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BEAMS WITHOUT LATERAL SUPPORT

By S. Timoshenko,* Esq.

Synopsis

An I-beam, without lateral support, under a load acting in the plane of the web, may become unstable and buckle sidewise. This is analogous to the buckling of a strut, when compressed by a force exceeding a limit given by well known column formulas.

Various approximate formulas are used in designing beams without lateral support. Such formulas are easily derived by considering the stability of the upper compressed flange of the beam and applying the column formula.

A more accurate method of solution of this problem, however, is developed in this paper. The sidewise buckling of I-beams under different loadings and different kinds of end fastenings is considered. In several cases, the critical values of maximum stresses are calculated and the results tabulated. By the use of these tables, the necessary reduction in working stresses can be easily calculated.

The general method used in the solution of the problem is outlined in Appendix I and, in Appendix II, the method of calculating critical loads beyond the elastic limit is shown.

The calculations show that the actual safety of I-beams without lateral supports is less than that indicated by the approximate formulas now in use, and that the ultimate strength of such beams depends not only on the ratio of length of beam to width of flange, as follows from approximate formulas, but also on the ratio of span to height of beam and on the position of loads. The beam obviously is less stable when the load acts on the upper flange than when it acts on the lower flange.

Introduction

It is well known that, in the absence of lateral supports, I-beams, bent in the plane of the web, may prove to be insufficiently stable. If the loads are increased beyond certain limits, such beams buckle sidewise and, becoming unable to support further loading, cause the structure to collapse.†

In designing beams without lateral support, practical engineers use various approximate formulas,‡ derived in a simple manner, by considering the stability

Note.—Written discussion on this paper, which will not be presented at any meeting of the Society, will be closed with the **August, 1924**, *Proceedings*. When finally closed, the paper, with discussion, will be published in *Transactions*.

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† An example of the destruction of girders in consequence of sidewise buckling is given by the bridge disaster near Tarbes, France, on July 17, 1897, *La Revue Technique*, November 15, 1897.

‡ "Beams Without Lateral Support", by R. Fleming, *Engineering News*, v. 75, No. 14 (1916).

of the upper compressed flange of the beam. As a result, the safe stress obtained depends only on the ratio of the length of the span to the breadth of the flange of the beam. A more exact solution* shows that the stability varies also with the ratio of the span to the height of the beam, and with the position of loads, as on the upper or the lower flange. The elevation of the point of application of a load is always connected with the decrease of stability of the I-beam and demands a suitable reduction of working stress.

Definitions and Symbols

In this paper, that magnitude of the load for which the plane form of bending of the I-beam becomes unstable and sidewise buckling begins, is called the critical load (Q_{cr}). The corresponding maximum stress is called the critical stress (p_{cr}).

The principal symbols are:

$2\,l$ = the length of beam.
h = the height of the web of beam.
I_1 = moment of inertia of beam about a gravity axis perpendicular to the web.
I_2 = moment of inertia of beam about a gravity axis in the plane of the web.
$B_1 = E\,I_1$ = the flexural rigidity of beam in bending in the plane of the web.
$B_2 = E\,I_2$ = the flexural rigidity of beam in the direction perpendicular to the web.
C = torsional rigidity by twisting of I-beam. (See page 241.)
p_{cr} = critical stress.
p'_{cr} = critical stress for load on upper flange.
p''_{cr} = critical stress for load on lower flange.

The following denotations are also used:

$$\alpha = \frac{C}{B_2}\left(\frac{2\,l}{h}\right)^2 \quad \ldots\ldots (1)$$

$$\beta = \frac{B_2}{B_1}\left(\frac{h}{2\,l}\right)^2 \quad \ldots\ldots (2)$$

Elements of I-Beam Sections.—In all cases to be considered, the critical load can be represented by the equation (see Appendix I):

$$Q_{cr} = K\,.\,\frac{\sqrt{B_2\,C}}{(2\,l)^2} \quad \ldots\ldots (3)$$

in which, K is a numerical factor depending on the magnitude of the quantity, α, given by Equation (1), on the distribution of load, and on the manner of fastening the ends of the beam.

* "Sur la Stabilité des Systèmes Elastiques", by S. Timoshenko, *Annales des Ponts et Chaussées*, 1913.

The magnitude of B_2 can easily be calculated from the cross-section of the beam. For example, in the case of a cross-section, as shown in Fig. 1, the approximate formula*:

$$B_2 = 2\,E\,\frac{\delta\,b^3}{12} \qquad \ldots\ldots(4)$$

can be used.

In calculating the torsional rigidity, C, for the same cross-section, the approximate formula:

$$C = G\left(\frac{2}{3}\,b\,\delta^3 + \frac{1}{3}\,h\,\delta_1{}^3\right) \qquad \ldots\ldots(5a)$$

in which, G denotes the modulus of rigidity,† can be used. This formula is obtained by the assumption that the torsional rigidity of an I-shaped cross-section (Fig. 1) is approximately equal to the sum of torsional rigidities of two narrow rectangles, $\delta \times b$, and a narrow rectangle, $\delta_1 \times h$. The torsional rigidity of a narrow rectangle, $\delta \times b$, can be taken as equal to $\frac{b\,\delta^3\,G}{3}$.

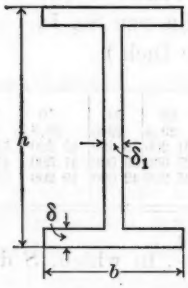

FIG. 1.

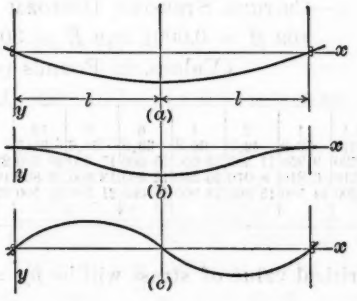

FIG. 2.

For more complicated cross-sections, C will be calculated by the use of the approximate equation of Saint-Venant:

$$C = \frac{1}{40} \cdot \frac{A^4}{I_p} \cdot G \qquad \ldots\ldots(5b)$$

in which, A denotes the area of the cross-section and I_p the polar moment of inertia of the cross-section, that is, $I_p = \frac{B_1 + B_2}{E}$.

METHODS OF SOLUTION

Consider, first, a beam simply supported at the ends, having a load uniformly distributed along the axis. In this paper, the phrase "simply supported", means that the ends of the beam can rotate freely about the y-axis and z-axis (see Fig. 4), but are restrained from rotating about the x-axis of the beam.

* δ_1 is considered as very small and the term, $\frac{(h-2\,\delta)\,\delta_1{}^3}{12}$, is neglected.

† In the calculations, G is taken equal to $0.4E$.

If this load exceeds a certain limit, the plane form of bending of the I-beam becomes unstable and sidewise buckling occurs. The buckled form of the beam is as shown in Fig. 4 and the lateral deflection of the central line as in Fig. 2 (*a*).

It will be noted that, owing to the type of end fastening, sidewise buckling of the beam is accompanied by twist. This explains why the stability of the beam depends not only on the lateral flexural rigidity, B_2, but also on the torsional rigidity, C, as shown by Equation (3).

The critical value of the load may be determined if the magnitude of the coefficient, K, in Equation (3) is known. This coefficient can be calculated by the use of the general method, explained in Appendix I. The results of these calculations are shown in Table 1. The values of the coefficient, K, are given as a function of the quantity, α, determined by Equation (1). If the dimensions of the I-beam are known, the values of C and B_2 can be calculated, as explained previously. Then, α will be found from Equation (1) and the corresponding value of K, obtained from Table 1. Substituting the value of K in Equation (3), the critical load, Q_{cr}, is obtained.

TABLE 1.—Critical Stresses, Uniform Load, in Terms of the Constant, α, for $\beta = 0.0001$ and $E = 30\,000\,000$ Lb. per Sq. In.
(Values, in Pounds per Square Inch.)

α =	0.1	1	2	4	6	8	12	16	20	32	50	70	90	100
k =	143.0	53.0	42.6	36.3	33.8	32.6	31.5	30.5	30.1	29.4	29.0	28.8	28.6	28.6
p_{cr} =	8 520	9 950	11 300	13 600	15 600	17 300	20 300	23 000	25 200	31 200	38 600	45 300	51 000	53 700
p'_{cr} =	5 510	6 810	8 070	10 300	12 200	13 800	16 800	19 400	21 600	27 600	35 000	41 600	47 400	50 000
p''_{cr} =	13 200	14 500	15 800	18 000	20 000	21 500	24 500	27 200	29 400	35 300	42 600	49 200	55 100	57 600

The critical value of stress will be $p_{cr} = \dfrac{Q_{cr} \cdot 2\,l}{8\,S}$, in which, S denotes the section modulus of the cross-section.

Taking into account that $S = \dfrac{2\,B_1}{E \cdot h}$ and using Equations (2) and (3), it is found that,

$$p_{cr} = \frac{K\,E}{16}\,\beta\,\sqrt{\alpha} \qquad (6)$$

Line 3 of Table 1 gives the values of p_{cr}, calculated from Equation (6), on the assumption that $\beta = 0.0001$ and $E = 30\,000\,000$ lb. per sq. in. From Table 1, the critical stresses for a beam with any other magnitudes of β and E_1 can easily be obtained. As seen from Equation (6), it is necessary only to multiply the corresponding number in Table 1 by $10^4\,\beta\,\dfrac{E_1}{E}$. The use of this method will now be shown by numerical examples.

Examples

Consider a structural beam of the following dimensions:

Length $= 2\,l = 20$ ft.
Depth $= h = 24$ in.

Flange width = $b = 7$ in.
Thickness of web = $\delta_1 = 0.5$ in.
Mean thickness of flanges = $\delta = \frac{1}{2}(0.60 + 1.14) = 0.87$ in.
Area of section = $A = 23.3$ sq. in.
Principal rigidity = $B_1 = 2\,087\,E$ lb. $\times$ sq. in.
" " = $B_2 = 42.7\,E$ lb. $\times$ sq. in.

Using Equation (5), the following is obtained:

$$C = \frac{23.3^4}{40\,(2\,087 + 42.7)} \cdot G = 3.46\,G.$$

Then, from Equation (1):

$$\alpha = \left(\frac{3.46 \times 0.4}{42.7}\right) 100 = 3.24$$

and from Equation (2):

$$\beta = \frac{42.7}{2\,087 \times 100} = 205 \times 10^{-6}.$$

Table 1 gives, by interpolation, for $\alpha = 3.24$,

$$p_{cr} = 11\,300 + \frac{1}{2}(13\,600 - 11\,300)\,1.24 = 12\,700 \text{ lb. per sq. in.}$$

This is the critical stress for $\beta = 0.0001$. The critical stress in the example considered will be $12\,700 \times \beta \times 10^4 = 26\,000$ lb. per sq. in. The load corresponding to this stress must be considered as the ultimate for the beam. From this numerical result it is noted that sidewise buckling occurs at stresses far less than the ultimate stress of the material under direct compression and even less than the elastic limit. This fact must be considered and the working stress lowered accordingly. Assuming, for instance, a factor of safety of 3, the working stress will be equal to $\frac{26\,000}{3} = 8\,670$ lb. per sq. in.

In calculating torsional rigidity, Equation (4) gives: $C = 4.07\,G$. Then, $\alpha = 3.80$. Table 1 gives:

$$p_{cr} = 11\,300 + \frac{1}{2}(13\,600 - 11\,300)\,1.80 = 13\,400 \text{ lb. per sq. in.}$$

Therefore, the critical stress in this case will be: $13\,400 \times \beta \times 10^4 = 27\,400$ lb. per sq. in. which is about 5½% greater than the result obtained previously. It will be noted that a considerable error in the magnitude of C has comparatively small influence on the critical stress. Therefore, approximate Equations (4), (5a), and (5b), can be used with sufficient accuracy in the calculations.

The magnitude of the critical stresses will now be determined by applying the approximate method of calculation, the upper compressed flange of the beam being considered as a column. On account of the fact that the compressive force in the flange is proportional to the bending moment and follows the parabolic law, the reduced length, $L = 0.694 \times 2\,l$, instead of the length, $2\,l$, must be substituted in the column formula. In the case con-

sidered, $L = 0.694 \times 20 \times 12 = 167$ in. The radius of gyration of the cross-section is $r = \frac{7}{2\sqrt{3}} = 2.02$ in. Therefore, $\frac{L}{r} = 83$.

For mild steel columns with pin ends, the straight-line formula gives:*

$$p_{cr} = 52\,500 - 220\,\frac{L}{r} = 52\,500 - 220 \times 83 = 34\,300 \text{ lb. per sq. in.}$$

which is a value 32% greater than that obtained by the use of the more accurate formula.

As a second example, consider a riveted plate girder consisting of a web-plate, 26 by $\frac{3}{8}$ in., and four flange angles, 5 by $3\frac{1}{2}$ by $\frac{1}{2}$ in.† The dimensions of the beam are:

$$\text{Length} = 2\,l = 480 \text{ in.}$$
$$\text{Depth} = h = 26\tfrac{1}{2} \text{ in.}$$
$$\text{Area of section} = A = 26 \times \tfrac{3}{8} + 4 \times 4 = 25.75 \text{ sq. in.}$$
$$\text{Principal rigidity} = B_1 = \left\{ \frac{3}{8} \times \frac{26^3}{12} + 4\,[4 + 4\,(13\tfrac{1}{4} - 0.91)^2] \right\}$$
$$= (550 + 2\,450)\,E = 3\,000\,E.$$
$$\text{Principal rigidity} = B_2 = 4\,[10 + 4\,(1.66 + 0.19)^2]\,E = 95\,E.$$

From Equation (4), the torsional rigidity is:

$$C = \frac{1}{40} \times \frac{25.75^4}{3\,095}\,G = 3.55\,G.$$

Substituting in Equations (1) and (2):

$$\alpha = \frac{3.55 \times G}{95 \times E} \times 328 = 4.91.$$

$$\beta = 0.967 \times 10^{-4}.$$

From Line 3 of Table 1, by interpolation:

$$p_{cr} = 13\,600 + \frac{(15\,600 - 13\,600)}{2} \times 0.91 = 14\,500 \text{ lb. per sq. in.}$$

In the case considered:

$$p_{cr} = 14\,500 \times \beta \times 10^4 = 14\,000 \text{ lb. per sq. in.}$$

When a uniform load is distributed along the upper flange of the beam, the same Equation (3) for Q_{cr} should be used, but, in accordance with decrease in stability, the values of the coefficient, K, become smaller. The corresponding values of critical stresses, p'_{cr}, for the case in which $\beta = 0.0001$, $E = 30\,000\,000$ lb. per sq. in., are given in Line 4 of Table 1. Line 5 of Table 1 furnishes the values of critical stresses, p''_{cr}, when the uniform load is distributed along the lower flange of the beam.

In order to show more clearly how the critical stresses decrease with an increase of the ratios, $\frac{2\,l}{h}$ and $\frac{h}{b}$, consider, as an example, the I-section

* "Applied Mechanics", Fuller and Johnston, 1919, p. 361.
† Pocket Companion, Carnegie Steel Co. (1920), p. 274.

(Fig. 1), in which, $\frac{\delta}{\delta_1} = 2$ and $\frac{b}{\delta} = 10$. The results of the calculations are given in Table 2. The variation of p_{cr} is also represented in Fig. 3 by Curves I, II, and III. The points corresponding to $\frac{2\,l}{b}$ = constant, are connected by dotted lines. It will be noted that, for a constant magnitude of the ratio, $\frac{2\,l}{b}$, the critical stress decreases with the increase of the depth of the beam.

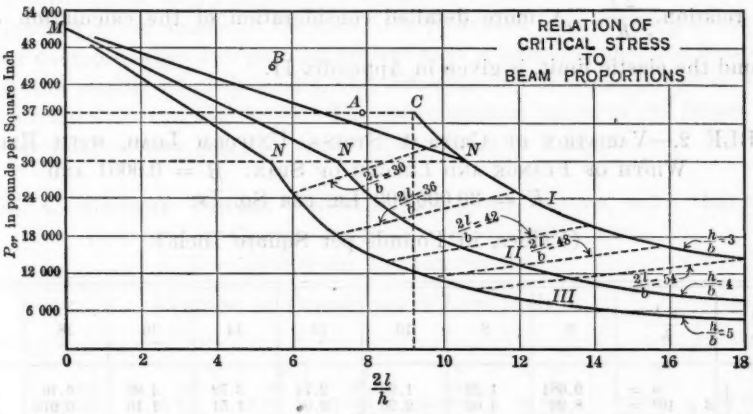

FIG. 3.

For comparison, the values of p_{cr}, calculated from the column formula, as explained previously, are as follows:

$2\,\frac{l}{b}=$	54	48	42	36
$p_{cr}=$	17 800	22 700	30 300	33 500

It will be noted that these stresses are always greater than those given in Table 2. For instance, if $\frac{h}{b} = 3$, the corresponding stresses from Table 2 are 14 400, 16 800, 20 000, and 25 100 lb. per sq. in., that is, the differences vary from 24 to 52 per cent. With increase of the ratio, $\frac{h}{b}$, the discrepancy increases between the figures of Table 2 and those obtained from the column formula.

STRESSES BEYOND THE ELASTIC LIMIT

Equation (3) is based on the assumption of perfect elasticity for the material of the beam; therefore, the critical stresses obtained from Table 1, or Fig. 3, represent the true values of these stresses only in case their magnitude is not greater than the elastic limit of the material. Otherwise, the critical stresses obtained in this manner will be too large. In order to obtain

the critical stress beyond the elastic limit, apply the straight-line formula which is used in the calculation of columns (Tetmayer's or Johnson's formulas). Assuming, for instance, 30 000 lb. per sq. in. as the elastic limit for structural steel, one finds that Curves I, II, and III, of Fig. 3, can be used only below the points, N. For higher stresses the straight lines, MN, should be used. (The ultimate compressive strength of the material is assumed to be 51 000 lb. per sq. in.). Curves analogous to the curves of Fig. 3 can easily be constructed from Table 1, for any cross-section of I-beam and in such a manner that the problem of critical stresses will be solved for any value of the relation, $\frac{2\,l}{h}$. A more detailed consideration of the calculation of p_{cr} beyond the elastic limit, is given in Appendix II.

TABLE 2.—Variation of Critical Stress, Uniform Load, with Relative Width of Flange and Length of Span. $\beta = 0.0001$ and $E = 30\,000\,000$ Lb. per Sq. In.

(Values, in Pounds per Square Inch.)

$\frac{h}{b}$	$\frac{2\,l}{h}$	6	8	10	12	14	16	18	20
3	α =	0.684	1.22	1.90	2.74	3.72	4.86	6.16	7.60
	$\beta \times 10^4$ =	8.23	4.63	2.96	2.06	1.51	1.16	0.915	0.741
	p_{cr} =	77 700	47 400	33 000	25 100	20 000	16 800	14 400	12 500
	p'_{cr} =	52 200	32 700	23 600	18 300	15 000	12 900	11 300	10 000
	p''_{cr} =	116 000	68 300	46 200	34 200	26 700	21 900	18 300	15 600
4	α =	0.720	1.28	2.00	2.88	3.92	5.12	6.48	8.00
	$\beta \times 10^4$ =	4.34	2.44	1.56	1.09	0.798	0.611	0.488	0.391
	p_{cr} =	41 300	25 200	17 600	13 400	10 800	9 000	7 730	6 750
	p'_{cr} =	27 900	17 600	12 600	9 840	8 120	6 930	6 080	5 420
	p''_{cr} =	61 100	36 300	24 600	18 300	14 300	11 700	9 810	8 390
5	α =	0.756	1.34	2.10	3.02	4.12	5.38	6.80	8.40
	$\beta \times 10^4$ =	2.61	1.47	0.940	0.650	0.480	0.368	0.290	0.235
	p_{cr} =	24 900	15 300	10 700	8 250	6 570	5 370	4 710	4 130
	p'_{cr} =	16 800	10 700	7 680	5 970	4 970	4 280	3 740	3 330
	p''_{cr} =	36 900	21 900	14 900	11 000	8 690	7 130	5 970	5 120

Effect of Restraint of Beams

For calculating critical loads for other methods of end fastening of beams, Equation (3) retains its form. The numerical factor, K, must be suitably changed, however. Numerical results of practical interest are given subsequently.

(*a*).—*Ends Built In.*—When a beam with built-in ends is buckled sidewise, the deflection line has the form represented in Fig. 2 (*b*). As a result of such end fastening, the stability of the beam increases and the factor, K, in Equation (3) also increases. Numerical values of this factor are given in Table 3. The values of p_{cr}, calculated on the assumption that $E = 30\,000\,000$ lb. per sq. in. and $\beta = 1 \times 10^{-4}$, are also given. For other values of E and β, the procedure is the same as that explained previously.

TABLE 3.—CRITICAL STRESS, UNIFORM LOAD, FOR BEAMS WITH ENDS BUILT IN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\alpha =$	0.1	1	2	4	8	24	32	50	100
$K =$	488	161	119	91.3	73.0	58.0	55.8	53.5	51.2
$p_{cr} =$	19 300	20 100	21 100	22 800	25 800	35 500	39 500	47 200	64 000

(*b*).—*Beams Laterally Supported at Center of Span.*—When a beam is laterally supported at the middle of the span, it is prevented from rotating about its axis (for example, when two parallel girders have a lateral connection between them at the middle of their span), the deflection by sidewise buckling will have the form shown in Fig. 2 (*c*). The corresponding values of the factor, K, and the critical stresses, p_{cr}, p'_{cr}, and p''_{cr}, are given in Table 4.

TABLE 4.—CRITICAL STRESS, UNIFORM LOAD, FOR BEAMS WITH LATERAL SUPPORT AT CENTER OF SPAN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\alpha =$	0.1	1	2	4	8	24	32	50
$K =$	673	221	164	126	101	79.5	76.4	72.8
$p_{cr} =$	39 900	41 600	43 500	47 000	53 300	72 900	81 000	96 600
$p'_{cr} =$	34 800	36 500	38 400	42 000	48 300	67 800	75 900	91 500
$p''_{cr} =$	45 900	47 300	49 200	52 800	59 100	78 600	86 600	102 000

CONCENTRATED LOADS

Equation (3) also holds true for a concentrated load.

(*a*).—*Beam Simply Supported.*—The corresponding formula for critical stress, when a critical load, P_{cr}, is acting at the middle of the span and the ends of the beams are simply supported, will be:

$$p_{cr} = \frac{P_{cr}\,(2\,l)}{4\,S} = \frac{K\,E}{8}\,\beta\,\sqrt{\alpha} \qquad (6a)$$

The corresponding values of the factor, K, and of the critical stresses, p_{cr}, p'_{cr}, and p''_{cr}, are given in Table 5.

TABLE 5.—CRITICAL STRESSES, CONCENTRATED LOADS, IN TERMS OF THE CONSTANT, α, FOR $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

$\alpha =$	0.1	1	2	4	6	8	12	16	20	24	40	60	80	100
$K =$	86.4	31.9	25.6	21.8	20.3	19.6	19.0	18.3	18.1	17.9	17.5	17.4	17.2	17.2
$p_{cr} =$	10 200	12 000	13 700	16 400	18 800	20 700	24 300	27 500	30 300	32 900	41 600	50 400	57 900	64 500
$p'_{cr} =$	6 080	7 580	9 000	11 600	13 800	15 800	19 200	22 400	25 100	27 600	36 300	45 000	52 500	59 100
$p''_{cr} =$	17 300	18 800	20 300	22 800	25 100	27 000	30 500	33 600	36 300	38 700	47 400	56 100	63 600	70 200

A comparison of the values of p_{cr}, given in Tables 1 and 5, shows that the ratio between them is nearly constant and equal to 1.2.

(*b*).—*Beam with Built-In Ends.*—For a beam with built-in ends, Fig. 2 (*b*), $p_{cr} = \frac{K E}{16} \cdot \beta \sqrt{\alpha}$. The values of the factor, K, and of the critical stress, p_{cr}, are given in Table 6.

TABLE 6.—CRITICAL STRESS, CONCENTRATED LOAD, FOR BEAMS WITH ENDS BUILT IN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

α =	0.1	1	2	4	6	8	10	12	16	32	50	80
K =	268	88.8	65.5	50.2	43.6	40.2	37.8	36.3	34.1	30.7	29.4	28.4
p_{cr} =	15 900	16 600	17 400	18 700	20 100	21 300	22 500	23 500	25 600	32 500	39 000	47 700

(*c*).—*Beam with Lateral Support at Center of Span.*—The values of the factor, K, and of the critical stresses, p_{cr}, for a beam having lateral support at the middle of the span, (Fig. 2 (*c*)), are given in Table 7.

TABLE 7.—CRITICAL STRESS, CONCENTRATED LOAD, FOR BEAMS WITH LATERAL SUPPORT AT CENTER OF SPAN. $\beta = 0.0001$ AND $E = 30\,000\,000$ LB. PER SQ. IN.

(Values, in Pounds per Square Inch.)

α =	0.1	1	2	4	8	24
K =	466	154	114	86.4	69.2	54.5
p_{cr} =	55 200	57 600	60 200	64 800	73 500	100 000

APPLICATION

By the use of Tables 1 to 7, approximate calculations of critical stresses may be made for the more complex cases. For example, in calculating the critical stress for several concentrated loads acting at the middle third of the span, Tables 5 to 7 may be used. If concentrated loads are distributed throughout the span, Tables 1 to 4, would be used. After the critical stress has been calculated, suitable reduction for working stress must be made.

APPENDIX I

DERIVATION OF GENERAL FORMULAS

The method used for obtaining the general equation for the critical load, $Q_{cr} = \frac{K \sqrt{B_2 C}}{(2\,l)^2}$, will be illustrated by a simple example. Suppose a strip, $A\,B$, Fig. 4 (*a*), of a narrow, rectangular cross-section is bent by a vertical concentrated force, P, acting in the vertical plane of symmetry through the

axis of the strip. If this force is small, the deflection of the beam will occur in the same vertical plane and this plane form of bending will be a stable one, that is, if an occasional force should produce a sidewise deflection, such deflection would disappear with the removal of the lateral force and the strip would return to its plane form of bending.

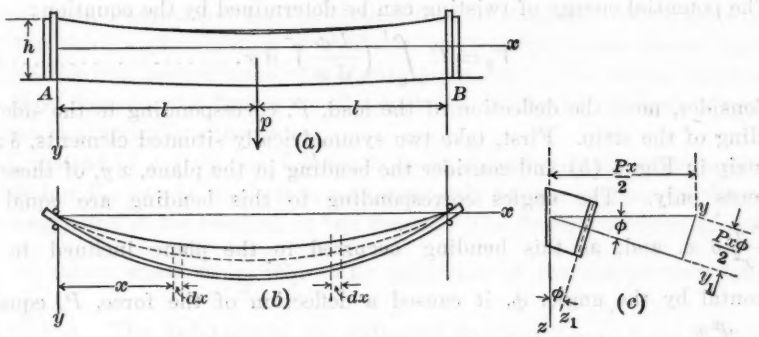

FIG. 4.

By increasing P, a limiting value of this force is reached at which the plane form of bending becomes unstable. The strip then buckles sidewise and large lateral deflections can occur without further increase of the force, P. This limiting value of the load is called the critical load. For its calculation, the method founded on the consideration of the potential energy of the system will be applied. Any lateral deflection of the strip will be accompanied by an increase of the potential energy of deformation. To the potential energy of bending in the plane of the strip* must be added the potential energy of bending in the lateral direction and the potential energy of twisting, both of which are functions of the lateral deflection. At the same time, the force, P, performs some additional work, as the sidewise buckling is accompanied by an increase of deflection in the direction of the force. Let T denote this additional work of the force, P, and V_1 and V_2, respectively, the potential energy due to bending in the lateral direction and to twisting.

If $V_1 + V_2 > T$, the lateral deflection is accompanied by an increase of the potential energy of the system, that is, the plane form of bending is a stable one. If $V_1 + V_2 < T$, the plane form of bending is unstable. The critical value of the force will be found from the equation:

$$V_1 + V_2 = T \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

From Fig. 4, it will be noted that the bending moment about the axis, Z_1, is equal to $\frac{1}{2} P x \phi$, in which, ϕ denotes the small variable angle of twist (Fig. 4 (c)). The differential equation for lateral deflection will be:

$$B_2 \frac{d^2 y}{d x^2} = -\frac{1}{2} P x \phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

* When the flexural rigidity, B_1, is great in comparison with B_2, it can be assumed that the potential energy of bending in the plane of web remains unchanged at buckling.

[If ϕ is a known function of x, the lateral deflection of the strip can be obtained from Equation (8).] The corresponding expression for potential energy will be:

$$V_1 = B_2 \int_0^l \left(\frac{d^2 y}{d x^2}\right)^2 d x = \frac{P^2}{4 B_2} \int_0^l x^2 \phi^2 d x \ldots\ldots\ldots (9)$$

The potential energy of twisting can be determined by the equation:

$$V_2 = C \int_0^l \left(\frac{d \phi}{d x}\right)^2 d x \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Consider, now, the deflection of the load, P, corresponding to the sidewise buckling of the strip. First, take two symmetrically situated elements, δx, of the strip in Fig. 4 (b) and consider the bending in the plane, $x y$, of these two elements only. The angles corresponding to this bending are equal to* $-\frac{d^2 y}{d x^2} \delta x$, and, as this bending occurred in the plane inclined to the horizontal by the angle, ϕ, it caused a deflection of the force, P, equal to $-x \phi \frac{d^2 y}{d x^2} \delta x$. Taking into account the bending of all elements of the buckled strip, the additional deflection in the direction of the force, P, is:

$$f = -\int_0^l x \phi \frac{d^2 y}{d x^2} d x$$

or, using Equation (8):

$$f = \frac{P}{2 B_2} \int_0^l x^2 \phi^2 d x$$

The corresponding work of the force, P, will be:

$$T = \frac{P^2}{2 B_2} \int_0^l x^2 \phi^2 d x \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

Substituting Equations (9), (10), and (11), in Equation (7),

$$P^2 = \frac{4 B_2 C \int_0^l \left(\frac{d \phi}{d x}\right)^2 d x}{\int_0^l x^2 \phi^2 d x} \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Substituting in Equation (12) for ϕ a suitably chosen function of x, satisfying the end conditions, the approximate value of the critical load can be calculated.

Assuming, for instance,

$$\phi = a \sin \frac{\pi x}{2 l}$$

there is obtained from Equation (12),

$$P_{cr} = \frac{17.2 \sqrt{B_2 C}}{(2 l)^2} \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

* If the angle, ϕ, is assumed to be small, the curvature in the plane, $x y$, can be taken equal to the curvature in the plane, $x y_1$.

The general expression for the angle, ϕ, in the case of symmetry,* can be taken in the form of the following trigonometrical series:

$$\phi = a_1 \sin \frac{\pi x}{2 l} + a_2 \sin \frac{3 \pi x}{2 l} + a_3 \sin \frac{5 \pi x}{2 l} + \ldots\ldots\ldots\ldots (14)$$

Using the two first members only of Equation (14) and choosing the coefficients, a_1 and a_2, in such a manner as to make Equation (12) a minimum, the second approximation for the critical load is obtained. With three members, the third approximation is obtained, etc. Calculations showed that the error of the first approximation is about 1½ per cent. The second approximation gives four correct figures for P_{cr}, and the calculation of further approximations is practically unnecessary.

If, instead of a strip, an I-beam is to be considered, the same method of calculating P_{cr} can be used; it is necessary only to take account of the potential energy due to the bending of the flanges. The flexural rigidity of a flange can be taken equal to $\frac{1}{2} B_2$. The deflection of the compressed flange is $y + \frac{1}{2} h \phi$. The deflection of the extended flange is $y - \frac{1}{2} h \phi$. Therefore, the potential energy of bending, due to the sidewise buckling, can be represented as follows:

$$V_1 = \frac{1}{2} B_2 \int_0^l \left(\frac{d^2 y}{d x^2} + \frac{1}{2} h \frac{d^2 \phi}{d x^2} \right)^2 d x + \frac{1}{2} B_2 \int_0^l \left(\frac{d^2 y}{d x^2} - \frac{1}{2} h \frac{d^2 \phi}{d x^2} \right)^2 d x$$

$$= B_2 \int_0^l \left(\frac{d^2 y}{d x^2} \right)^2 d x + \frac{1}{2} B_2 h^2 \int_0^l \left(\frac{d^2 \phi}{d x^2} \right)^2 d x \ldots\ldots\ldots (15)$$

Equation (15), instead of Equation (9), must be substituted in Equation (7) to calculate P_{cr} for an I-beam. The expression for P_{cr} retains its previous form:

$$P_{cr} = \frac{K \sqrt{B_2 C}}{(2 l)^2}$$

The values of K, obtained by retaining the two first members of Equation (14), are given in Table 5. The same method was used also in the calculation of all the other tables.

APPENDIX II

Derivation for Conditions Beyond the Elastic Limit

The calculation of a critical load beyond the elastic limit is possible if the relation between the strain, e, and corresponding stress, p, for unelastic deformations is known. This relation is no longer a linear one, and the

* As in the case of the compressed column many different buckled forms of equilibrium may be obtained in the case considered, but the first one only, which is symmetrical, Fig. 4(b), is of practical interest.

modulus of elasticity, E_1, beyond the elastic limit, will be variable. It may be defined as follows:

$$\text{Modulus of elasticity beyond the elastic limit} = E_1 = \frac{\delta p}{\delta e} \quad \ldots\ldots(16)$$

in which δe is the increase of strain corresponding to an increase of stress equal to δp.

From strain-stress diagrams of material beyond the elastic limit (strain being the horizontal co-ordinate), the quantity, E_1, for any value of p, will be equal to the slope of the curve at the point, p.

Consider the case of structural steel for which $E = 30\,000\,000$ lb. per sq. in., and the limit of elasticity is 30 000 lb. per sq. in. Suppose that the modulus, E_1, beyond the elastic limit can be represented by the equation:

$$E_1 = \frac{(60\,000 - p)^2}{30} \quad \ldots\ldots(17)$$

that is, E_1 decreases according to a parabolic law. Some values of the ratio, $\frac{E_1}{E}$, calculated from Equation (17), are given in the second line of Table 8.

TABLE 8.—Relation of Moduli to Stress.

p, in pounds per square inch =	30 000	37 500	45 000	52 500	60 000
$\frac{E_1}{E}$ =	1	$\frac{9}{16}$	$\frac{1}{4}$	$\frac{1}{16}$	0
$\frac{E'}{E}$ =	1	$\frac{36}{49}$	$\frac{4}{9}$	$\frac{4}{25}$	0

From a consideration of the bending of beams compressed beyond their elastic limits, it is known that the flexural rigidity decreases with an increase of compressive stress and that, in the case of a rectangular cross-section, this rigidity can be calculated from the equation*:

$$B' = E' I \quad \ldots\ldots(18)$$

in which, I is the moment of inertia of the cross-section about the neutral axis, and,

$$E' = \frac{4 E E_1}{(\sqrt{E} + \sqrt{E_1})^2} \quad \ldots\ldots(19)$$

Some values of the ratio, $\frac{E'}{E}$, calculated from Equation (19), are given in Line 3 of Table 8.

In order to simplify the problem of the calculation of critical load, the following two assumptions will be made:

(1).—That the bending of beams beyond the elastic limit does not diminish their torsional rigidity, C.

(2).—That, in calculating the diminished value of flexural rigidity, B_2, the compressive and tensile stresses in the flanges are constant along the span

* T. Karman, "Untersuchungen über Knickfestigkeit", Göttingen, Dissert, 1909.

of the beam and equal to the maximum bending stress. Therefore, any diminution of B_2 will also be constant along the span of the beam. If, for instance, the maximum bending stress is equal to 37 500 lb. per sq. in., the rigidity in the lateral direction (Table 8) will be diminished along the whole span in the ratio, 36:49.

The latter assumption is more unfavorable than the actual conditions, because, in reality, the maximum stress occurs only in the cross-section of the maximum bending moment. In the other cross-sections, the stresses are smaller and, for this reason, the actual decrease of B_2 will not be constant along the entire span of the beam. It will now be shown how, on the assumptions made, and for an I-beam of a known cross-section, the ratio, $\frac{2\,l}{h}$, corresponding to any given value of critical stress beyond the elastic limit, may be obtained. As an example, consider a cross-section for which $\frac{\delta}{\delta_1} = 2$; $\frac{b}{\delta} = 10$; $\frac{h}{b} = 3$. The relation between critical stress and the ratio, $\frac{2\,l}{h}$, within the elastic limit is represented by Curve I in Fig. 3; the corresponding numerical values of critical stresses are given in Table 2.

Suppose, now, the critical stress lies beyond the elastic limit, as, for instance, that $p_{cr} = 37\,500$ lb. per sq. in. To this stress there corresponds on Curve I in Fig. 3, the point, C; and from that diagram, $\frac{2\,l}{h} = 9.2$. It is known, however, that Curve I gives exaggerated values of p_{cr} beyond the elastic limit. The true values of p_{cr} will be less, that is, a smaller value of the ratio, $\frac{2\,l}{h}$, than that found previously and will correspond to the stress, 37 500 lb. per sq. in. This true value of $\frac{2\,l}{h}$ can be found by successive approximations.

Assume, for instance, that this true value is $\frac{2\,l}{h} = 8.0$. For this value of $\frac{2\,l}{h}$, from Table 2, $\alpha = 1.22$, $\beta = 4.63 \times 10^{-4}$. Taking account of the fact that the lateral rigidity, B_2, decreases beyond the elastic limit and that this decrease for the stress, 37 500 lb. per sq. in., is (see Table 8), in the ratio, 36:49, it is found in this case that:

$$\alpha = 1.22 \times \frac{49}{36} = 1.66$$

$$\beta = 4.63 \times \frac{36}{49} \times 10^{-4} = 3.40 \times 10^{-4}$$

For these values of α and β, it is found from Table 1 that,

$$p_{cr} = [9\,950 + 0.66\,(11\,300 - 9\,950)]\,3.40 = 36\,900 \text{ lb. per sq. in.}$$

This is less than the critical stress, 37 500 lb. per sq. in., taken previously, that

is, the correct ratio, $\frac{2\,l}{h}$, is less than 8. As a second approximation, assume $\frac{2\,l}{h} = 7.90$; then,

$$\alpha = 1.66 \left(\frac{7.9}{8}\right)^2 = 1.62\,;\ \beta = 3.40 \left(\frac{8}{7.9}\right)^2 10^{-4} = 3.49 \times 10^{-4}$$

and, from Table 1,

$$p_{cr} = [9\,950 + (11\,300 - 9\,950) \times 0.62]\ 3.49 = 37\,600 \text{ lb. per sq. in.}$$

This result approaches the stress, 37 500 lb. per sq. in., taken previously and, therefore, the second approximation for the ratio, $\frac{2\,l}{h}$, is sufficiently exact. This solution is represented in Fig. 3, by the point, *A*.

As a second example, taking a beam of the same cross-section, the ratio, $\frac{2\,l}{h}$, corresponding to $p_{cr} = 45\,000$ lb. per sq. in., will be found. As a first approximation, let $\frac{2\,l}{h} = 6$. Then, from Tables 2 and 8,

$$\alpha = 0.684\,\frac{9}{4} = 1.54\,;\ \beta = 8.23 \times 10^{-4} \times \frac{4}{9} = 3.65 \times 10^{-4}$$

For these values of α and β, Table 1 gives,

$$p_{cr} = [9\,950 + (11\,300 - 9\,950)\ 0.54]\ 3.65 = 39\,000 \text{ lb. per sq. in.}$$

Hence, the value taken as the first approximation for the ratio, $\frac{2\,l}{h}$, is too large. As a second approximation, assume that $\frac{2\,l}{h} = 5.3$. Then, in the same manner as before,

$$\alpha = 1.54 \left(\frac{5.3}{6.0}\right)^2 = 1.20\,;\ \beta = 3.65 \times 10^{-4} \left(\frac{6.0}{5.3}\right)^2 = 4.67 \times 10^{-4}\,;$$

$$p_{cr} = 47\,700 \text{ lb. per sq. in.}$$

This value of p_{cr} is too large; therefore, as a third approximation giving a sufficiently exact result, try $\frac{2\,l}{h} = 5.48$. In Fig. 3, this result is represented by the point, *B*.

In a similar manner is found the value of $\frac{2\,l}{h}$ for every other value of p_{cr} beyond the elastic limit, and the corresponding curve representing the relation between p_{cr} and $\frac{2\,l}{h}$ may be constructed. This curve, as appears from the calculations, lies above the line, *MN*, of the straight-line formula previously proposed as an approximate solution, that is, the calculations with this formula will always give a somewhat higher coefficient of safety than the alternative straight-line solution.

ANALYSIS OF RAILROAD GRADES FOR CLASSIFICATION

By S. Vilar y Boy,* Assoc. M. Am. Soc. C. E.

Synopsis

Although all authorities dealing with railroad economics show distinctly the mathematical line of demarcation between Class A and Class B grades, used for the computation of operating expenses, the writer knows of only one table (by J. B. Berry, M. Am. Soc. C. E.†) that defines with numerical values the limit for Class B and Class C grades. This table gives figures which apply to the conditions of velocity limits and train resistance assumed in its computation, but leaves the question unsettled for any other case.

With this in mind, the writer has endeavored to solve the general problem of grade analysis mathematically, giving results to fit any actual condition by simply substituting values in a general formula which should lead to the data compiled by Mr. Berry, if his conditions are assumed; but, in any case, by comparison with an assumed standard of classification, the proper rating may be found to suit local track and operating conditions.

An attempt has been made to avoid the tedious and elaborate calculations which would make the method virtually useless in current practice. In addition, there have been included complementary problems involving curve resistance, momentum grades, and the use of sand, all of which are capable of solution on the same basis.

Introduction

The several lines constructed and under construction by the International Railways of Central America indicated to the writer the convenience of a definite basis for analyzing grades, showing whether any proposed alteration would be an improvement or an impairment, without leaving the question open to judgment, and especially bearing in mind that on mountain roads the changes of grade are rather frequent. This fact makes the classification much more haphazard than in the case of continuous and uniform grades, unless some numerical standard is established.

General Methods

Evidently, the starting point should be the differential equation of kinetic energy, taking into consideration the train resistance for various velocities.

Note.—Written discussion on this paper, which will not be presented at any meeting of the Society, will be closed with the **August, 1924,** *Proceedings*. When finally closed, the paper, with discussion, will be published in *Transactions*.

* Office Engr., International Rys. of Central America, Guatemala, Guatemala.

† "Economics of Revision Work", *Proceedings*, Annual Convention, 1904, American Railway Engineering Association.

The formula of the Baldwin Locomotive Works for train resistance, $R = 3 + \frac{v}{6}$, appeared to be suitable, but instead of using v in miles per hour for homogeneity of the kinetic energy equation, its value in feet per second should be used.

If v is expressed in feet per second in the Baldwin formula, it becomes:

$$R = \frac{\text{Velocity in feet per second}}{6 \times 1.47} + 3 = \frac{\text{Velocity in feet per second}}{8.82} + 3. \quad (1)$$

The parameter, 3, refers to a rather low track resistance, namely, $\frac{30}{6} + 3 = 8$ lb. per ton for 30 miles per hour. This value should be altered to suit actual conditions of track and equipment in any particular case, as may be ascertained by dynamometer tests.

If the train resistance is 10 lb. per ton, at 30 miles per hour $\left(\text{equal to } \frac{30}{6} + 5\right)$, the modified Baldwin formula should be:

$$R = \frac{\text{Velocity in feet per second}}{8.82} + 5 \quad (2)$$

Down-Grade Computations

For a train running down grade, the differential equation of kinetic energy is:

$$(f - f')\, ds = m\, v\, dv \quad (3)$$

in which,

f = grade acceleration (in this case 20 lb. times the percentage of gradient).

f' = train resistance, as previously explained.

dv = change of velocity in the distance, ds.

m = mass.

v = velocity.

Substituting these values and assuming, for simplicity, a 1% grade:

$$(f - f')\, ds = \left[20 - \left(\frac{v}{8.82} + 5\right)\right] ds = m\, v\, dv$$

Solving,

$$ds = \frac{8.82\, m\, v\, dv}{132.3 - v}$$

As,

$$m = \frac{2\,000}{32.16} = 62.2$$

$$s = 8.82 \times 62.2 \int_{v_0}^{v_1} \frac{v\, dv}{132.3 - v}$$

or,

$$s = 548.6 \int_{v_0}^{v_1} \frac{132.3 + v - 132.3}{132.3 - v}\, dv$$

whence,

$$s = 548.6 \left[132.3 \int_{v_0}^{v_1} \frac{dv}{132.3 - v} + \int_{v_0}^{v_1} - dv \right]$$

$$s = 548.6 \left[- 132.3 \text{ Napierian log } \Big(132.3 - v\Big)_{v_0}^{v_1} - (v_1 - v_0) \right]$$

Assuming the velocity limits of 30 and 10 miles per hour, as used by Mr. Berry, corresponding to 44.1 and 14.7 ft. per sec., respectively:

$$s = 548.6 \left[- 29.4 - 132.3 \times 2.3 \text{ (log } 88.2 - \text{log } 117.6) \right]$$

$$s = 4\,738 \text{ ft.}$$

as compared with 4 800 ft. in the table compiled by Mr. Berry.

If X = grade resistance, in pounds per ton (Baldwin parameter) $\times$ 8.82, the working formula can be shown, thus:

$$s = 548.6 \left[(v_0 - v_1) - X \text{ Napierian log } \Big(X - v\Big)_{v_0}^{v_1} \right] \ldots\ldots\ldots (4)$$

(Both the distance, s, and the velocities are in feet and the time is in seconds.)

For the cases taken by Mr. Berry, the results are as shown in Table 1.

TABLE 1.—DISTANCES IN WHICH A TRAIN, WITH STEAM SHUT OFF AND WITHOUT APPLYING BRAKES, SHOULD INCREASE ITS VELOCITY FROM 10 TO 30 MILES PER HOUR, AT DIFFERENT GRADES.

MOTION DOWN GRADE.

Percentage of gradient.		0.6	0.7	0.8	0.9	1.0	1.25	1.50
As calculated by Mr. Berry	Distance, in feet	17 000	10 000	7 300	5 700	4 800	3 300	2 600
	Fall, in feet	102	70	58	51	48	41	38
As calculated by the writer	Distance, in feet	17 054	10 233	7 375	5 760	4 738	3 236	2 512
	Fall, in feet	102.3	71.6	59.0	51.8	47.4	40.5	37.7

The close agreement between the calculations of Mr. Berry and the writer seems to warrant the extension of the method explained to cases of different velocity limits, train resistance, and grades, without going to the trouble and expense of experimenting.

It might be objected that, in the Baldwin formula, the resistance is assumed to be a straight-line function of the velocity, whereas it is well known that this is not true for any wide variation of velocity. Deductions based on this straight-line formula will give results approximately correct only within the proper ranges of velocity.

Although the figures obtained by Mr. Berry and by the writer seemed to endorse the Baldwin formula for this purpose, second-degree functions also can be used, as, for instance, in the following:

The Wellington typical formula:

$$R = 4 + 0.005\, V^2 + (0.28 + 0.3\, N) \frac{V^2}{W}$$

which, substituting values for N and W, reduces to:

$$R = 4 + \frac{V^2}{A} \ldots\ldots\ldots (5)$$

The Wellington short formula:

$$R = 4 + \frac{V^2}{130} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)^*$$

The D. K. Clark formula:

$$R = \frac{2\,000}{2\,240}\left(8 + \frac{V^2}{171}\right) = 7.14 + \frac{V^2}{192} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)^*$$

All these formulas have the same general form: $R = a + \frac{V^2}{b}$. The following derivation, on the basis of the Wellington short formula, will apply to all.

As already explained, for homogeneity of the kinetic energy equation, the velocity should be given in feet per second instead of miles per hour. This modifies the Wellington formula to read:

$$R = 4 + \frac{(\text{Velocity in ft. per sec.})^2}{1.47^2 \times 130} = 4 + \frac{(\text{Velocity in ft. per sec.})^2}{281} \quad \ldots\ldots (8)$$

The differential equation for the train running down a 1% grade should be:

$$\left[20 - \left(\frac{v^2}{281} + 4\right)\right] ds = m\, v\, dv$$

Solving, as before:

$$s = 8\,739.1 \left[\text{Napierian log } (4\,496 - v^2)\right]_{v_1}^{v_0}$$

In the case assumed:

$$v_0^2 = 14.7^2 = 216.1\ ;\ v_1^2 = 44.1^2 = 1\,944.8.$$

Therefore:

$$s = 8\,739.1 \times 2.3\ (\log 4\,279.9 - \log 2\,551.2) = 4\,518 \text{ ft.}$$

as compared with 4 800 ft. from the data given by Berry and 4 738 ft., as given by the Baldwin formula.

Applying the same Wellington formula to a 0.6% grade: $s = 16\,604$ ft., as compared with 17 000 ft. from the data compiled by Mr. Berry and 17 054 ft. from the Baldwin formula.

In both cases, the distances obtained by the Wellington equation are lower than the others, which indicates that the retardation assumed is too small.

In attempting to adjust the discrepancy, the writer increased the parameter, that is, the friction, in the Wellington formula from 4 to 4.5, with the following results: For a 1% grade: $s = 4\,719$ ft., which is still too low; and for a 0.6% grade: $s = 21\,410$ ft., which is exceedingly high compared with the Berry data. This compelled the writer to rely on the Baldwin formula as it agreed better with that of Berry, the only available authority.

Up-Grade Computations

When a train runs down grade, the resistance and grade acceleration are of opposite signs, whereas running up grade it meets resistance from both the grade and friction, thus shortening the distance traveled within the same velocity limits.

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* *Transactions*, Am. Soc. C. E., Vol. VIII (1879), pp. 21–54.

Under these conditions, it might happen that a rise of less than 30 ft. per mile, which, at first sight, indicates Class A, becomes Class B, requiring steam manipulation if it follows a descent calling for steam to be shut off. In order to define this point of view more clearly, Table 2 for "up grade" has been calculated to correspond with the foregoing "down-grade" statement. This merely requires a change of signs in Equation (3). For a 1% grade, as before:

$$\left[20 + \left(\frac{v}{8.82} + 5\right)\right] ds = m\, v\, dv$$

Solving,

$$ds = 8.82\, \frac{m\, v\, dv}{220.5 + v};\ s = 8.82 \times 62.2 \int_{v_0}^{v_1} \frac{v\, dv}{220.5 + v}$$

whence,

$$s = 548.6 \int_{v_0}^{v_1} dv - 220.5 \int_{v_0}^{v_1} \frac{dv}{220.5 + v} =$$

$$548.6 \left[(v_1 - v_0) - 220.5 \text{ Napierian log } (220.5 + v)\Big|_{v_0}^{v_1}\right]$$

As previously stated, the results may be tabulated as shown in Table 2.

TABLE 2.—Distances in Which a Train with Steam Shut Off and Without Applying Brakes Should Decrease Its Velocity from 30 to 10 Miles per Hour at Different Grades. Motion Up Grade.

Percentage of gradient.....	0.6	0.7	0.8	0.9	1.0	1.25	1.50
Distance, in feet.....	2 639	2 408	2 200	2 052	1 898	1 608	1 404
Rise, in feet.....	15.8	16.8	17.6	18.4	19.0	20.1	21.0

which, compared with the data for "down grade" (Table 1) shows the importance of considering the variable train resistance.

Letting $Y = 8.882 \times$ (grade resistance in pounds per ton + Baldwin parameter), the working formula for this case becomes:

$$s = 548.6 \left[(v_1 - v_0) - Y \text{ Napierian log } (Y + v)\Big|_{v_0}^{v_1}\right] \ldots\ldots\ldots (9)$$

Curve Resistance

As an extension to this method, the curve resistance may be taken into account by two different methods:

1.—If the generally accepted compensating rate, namely, 0.03 to 0.04 of 1% grade for every degree of curvature, is assumed, it is merely a matter of increasing or decreasing the grade resistance accordingly, without altering the integration.

2.—Otherwise, a variable curve resistance, such as that of Messrs. E. C. Schmidt and H. H. Dunn,* may be introduced thus:

$$\text{Curve resistance} = r = \alpha\, V \times C \ldots\ldots\ldots\ldots (10)$$

* *Bulletin 92*, Univ. of Illinois.

in which, V = speed, in miles per hour; C = degree of curve; and α = numerical constant.

Assume for simplicity that a tangent resistance of 10 lb. is doubled on a 10° curve. Then,

$$10 = \alpha \times 30 \times 10, \; \alpha = \frac{10}{300} = 0.0333.$$

Therefore,

$$r = 0.0333 \; V \; C$$

Again, if the velocity is given in feet per second:

$$r = \frac{0.0333}{1.47} \times \text{velocity in feet per second} \times C$$

Assuming a 1% down grade and an 8° curve, for instance, the differential equation of kinetic energy becomes:

$$\left[20 - \left(\frac{v}{8.82} + 0.184\, v + 5\right)\right] ds = \left(15 - \frac{v}{3.36}\right) ds = m\, v\, dv$$

As before, the solution of this expression involves Napierian logarithms.

Use of Sand

It is possible to determine mathematically the point where sand must be used by referring to the ratio of driver adhesion and train load. Take, for instance, the typical Consolidation engine assumed by Webb for his examples, and the adhesion ratio as 0.20 for a wet rail (or still lower if a further margin of safety is required). The available draw-bar pull is 10.6 tons and, if the ruling grade on the Division is 1% and the train resistance 10 lb., the maximum required pull is 30 lb. per ton and the maximum train load is 706 tons.

Let a fully loaded train approach, at 30 miles per hour, a 1.50% hump that follows a descent requiring steam to be shut off. It will be necessary to admit steam gradually, and the draw-bar pull will increase from zero at the bottom to 30 lb. per ton at the time sand is required. As the gradual opening of the throttle depends on the locomotive engineer, the result obtained should be considered as a tentative solution only, no matter how much mathematical accuracy is exercised.

Although it seems probable that the opening would be a straight-line function of the elapsed time, it is much easier to make it a function of the variable already involved in the differential equation, namely, the velocity. From this standpoint, the allowance of steam should be at the rate required to counteract the loss of kinetic energy at any moment, as compared with the total loss within the assumed velocity limits, thus:

$$\frac{\frac{m\,v^2}{2} - \frac{m\,v_0^2}{2}}{\frac{m\,v_1^2}{2} - \frac{m\,v_0^2}{2}} = \frac{v^2 - v_0^2}{v_1^2 - v_0^2} \qquad (11)$$

For practical purposes, the following equation might suffice:

$$f = P\left(1 - \frac{v^2 - v_0^2}{v_1^2 - v_0^2}\right) \qquad (12)$$

in which,

P = maximum available pull per ton;

v_1 = velocity, in feet per second, at the bottom of the grade;

v_0 = velocity, in feet per second, at the point where sand is required.

When $v = v_1$, $f = 0$; that is, no draw-bar pull is required at the bottom as no kinetic energy has yet been lost. When $v = v_0$, that is, at the point where sand is required, $f = P$, namely, the entire available pull.

Using the previous velocity limits.

$$f = 30\left(1 - \frac{v^2 - 14.7^2}{44.1^2 - 14.7^2}\right) = 33.75 - \frac{v^2}{57.62}$$

This being a second degree function of v, it should be substituted in the "up-grade" kinetic equation for a train climbing the 1.5% hump. Using the Wellington formula for an easier integration:

$$\left[\left(33.75 - \frac{v^2}{57.62}\right) - \left(30 + \left(\frac{v^2}{281} + 4\right)\right)\right] ds =$$

$$m\, v\, dv = -\left(0.25 + \frac{338.62}{16\,191.2}\, v^2\right) ds$$

whence,

$$ds = -\frac{47.81\, m\, v\, dv}{11.95 + v^2} \text{ and } s = 47.81 \times 62.2 \int_{v_1 = 44.1}^{v_0 = 14.7} -\frac{v\, dv}{11.95 + v^2}$$

$$= 1\,487 \times 2.3\,[\log (11.95 + 1\,944.8) - \log (11.95 + 216.1)] = 3\,182 \text{ ft.}$$

How near the operator's judgment would actually be to the assumption noted is difficult to say, but it seems reasonable that it should depend on the distance already traveled along the hump, because in the foregoing formula, distance is the factor involved in the kinetic energy equation:

$$f\, s = \frac{m\, v^2}{2} \qquad (13)$$

On the other hand, if the operator's judgment depends on the time elapsed, his throttle regulation should virtually agree with the loss of momentum at any time compared with the total loss within the assumed velocity limits; that is,

$$\frac{m\, v - m\, v_0}{m\, v_1 - m\, v_0} = \frac{v - v_0}{v_1 - v_0} \qquad (14)$$

as time is the factor involved in the momentum equation:

$$f\, t = m\, v \qquad (15)$$

In this case, the formula for variable draw-bar pull might better follow the form:

$$f = P\left(1 - \frac{v - v_0}{v_1 - v_0}\right) \qquad (16)$$

For $v_0 = 14.7$ and $v_1 = 44.1$:

$$f = 30\left(1 - \frac{v - 14.7}{29.4}\right) = 45 - 1.02\, v$$

Substituting this formula in the "up-grade" kinetic equation for a train climbing the 1.5% hump, as already outlined, and using the Baldwin straight-line formula to avoid elaborate integrals, inasmuch as the traction function is a straight line:

$$\left[(45 - 1.02\,v) - \left(30 + \left(\frac{v}{8.82} + 5\right)\right)\right] ds = m\,v\,dv = \left(10 - \frac{v}{0.882}\right) ds$$

which is the same kind of differential equation as that already used.

Solving,

$$s = 0.882 \times 62.2 \int_{v_1 = 44.1}^{v_0 = 14.7} \frac{v\,dv}{8.82 - v}$$

which equation would lead to logarithms of negative numbers. In order to avoid this, the integrable form should be:

$$s = 54.86 \int_{v_1}^{v_0} \frac{-v}{v - 8.82}\,dv =$$

$$54.86 \left[(v_1 - v_0) - 8.82 \text{ Napierian log } \left(v - 8.82\right)_{v_1}^{v_0}\right]$$

Substituting the previous velocity limits, $s = 2\,478$ ft., as compared with 3 182 ft. from the Wellington formula. The lower value, 2 478 ft., being more conservative and being derived from the Baldwin formula which agrees fairly well with the Berry figures, seems to be preferable.

Opponents of straight-line resistance formulas however, might take the average 2 830 ft. which involves a possible error of about 12% either way. This variation is not important in practice because of the uncertainty in the personal factor of operation.

If the straight-line resistance formula is used, with the second degree function for throttle opening, or *vice versa*, it reduces to an integral of the form:

$$\int_{v_1}^{v_0} \frac{v\,dv}{a + bv + c\,v^2} \qquad (17)$$

which is readily found in mathematical reference books, or may be integrated in series. Such mathematical refinement is unnecessary; any solution is only tentative.

Incidentally, it might happen that the integral, $\int_{v_1}^{v_0} \frac{v\,dv}{A - v}$, is of such numerical value that, $A - v = 0$, which leads to the indeterminate function, Napierian log, 0 as shown by the following example.

Consider a train climbing a 1% momentum grade, and endeavor to calculate the distance traveled at the time the engine has to exert its total tractive effort.

Train resistance on a 1% grade and at 10 miles per hour according to the modified Baldwin formula $= 20 + \left(\frac{10}{6} + 5\right) = 26.6666$. This being the maximum traction assumed, the gradual throttle opening will be represented by:

$$f = 26.6666 \left(1 - \frac{v - 14.7}{44.1 - 14.7}\right) = 40.016 - 0.908163\,v$$

Substituting,

$$\left[40.016 - 0.908163\, v - \left(20 + \frac{v}{8.82} + 5\right)\right] ds = m\, v\, dv$$

$$= \left[15.016 - \frac{v}{0.97891}\right] ds$$

and, finally,

$$s = 60.89 \Big\{ [44.1 - 14.7]$$

$$- 14.7\ [\text{Nap. log. } (14.7 - 14.7) - \text{Nap. log } (44.1 - 14.7)] \Big\}$$

the indeterminate function being Nap. log (14.7 — 14.7).

The mathematical explanation is that in the fundamental equation of kinetic energy, $f = 15.016 - \frac{v}{0.97891}$ = a retarding force expressed by a negative sign for all values of v from the initial velocity of 44.1 ft. per sec. assumed, to a velocity of 14.7 ft. per sec. when the force becomes zero and, consequently, the retardation becomes zero. Evidently, then, the mass approaches the velocity of 14.7 ft. per sec. as a minimum. In other words, a mass cannot be retarded from a higher to a lower velocity at a zero rate of retardation, and, naturally, the assumption of a velocity of 14.7 leads to an indeterminate condition.

Practically, inasmuch as the subtracting term, 0.97891×15.016, is derived from the assumed maximum pull, 26.6666, indeterminate results can be avoided by assuming a value of velocity which does not conflict with the minimum velocity previously selected.

Conclusion

Finally, it should be remembered that this paper is not written from the point of view of a scientist, but is simply intended to afford a handy means of computation, based on the experience of Mr. Berry, that will be suitable for current railroad office practice as regards economics of location which is generally accepted as a guide rather than as a strictly mathematical development. Furthermore, as the basis of the method explained is abstract mathematics, actual figures can be substituted for resistance parameters to suit the physical conditions.

SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION

FINAL REPORT ON SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF STEEL HIGHWAY BRIDGE SUPERSTRUCTURE

COMMITTEE

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February 4, 1924.

To the Board of Direction
American Society of Civil Engineers:

The Special Committee on Specifications for Bridge Design and Construction transmits herewith its Final Report on Specifications for Steel Highway Bridge Superstructure, as revised at its meeting in Chicago, Ill., February 1, 1924.

In accordance with your instructions these Specifications are recommended for adoption.

Henry B. Seaman,
Chairman.

February 4, 1924.

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SPECIFICATIONS FOR STEEL HIGHWAY BRIDGE SUPERSTRUCTURE

SECTION A.—GENERAL

1.—These specifications cover the design of fixed span bridges and require first-class design, material, and workmanship, subject to the approval of the Engineer.

2.—Material.—Bridges shall be made wholly of structural steel except where otherwise specified. Castings shall be of steel, unless cast iron is specifically authorized by the Engineer.

3.—Notice to Engineer.—The Contractor shall give ample notice to the Engineer of the beginning of rolling and of shop work, so that inspectors may be provided. No material shall be rolled, or work done, before the Engineer has been notified where the orders have been placed.

4.—Type of Bridge.—The type of bridge to be used for various span lengths preferably shall be as follows:

Rolled beams up to	40 ft.
Plate girders or lattice trusses	30 to 100 ft.
Riveted half-through trusses	50 to 100 ft.
Riveted trusses	90 ft. or over
Riveted or pin-connected trusses above	150 ft. or over

5.—Trusses.—Preference will be given to trusses with single intersecting web members, or other forms of trusses possessing the least ambiguity in computed stresses and the greatest elements of serviceability. Adjustable members in any part of the structure preferably shall be avoided. Members shall be symmetrical about the central planes of the trusses, and all parts shall be designed so that they can be inspected, cleaned, and painted. Through riveted and pin-connected spans will generally have inclined end posts. Half-through trusses shall be of the riveted type.

6.—Spacing of Trusses.—The width between centers of trusses or girders shall be sufficient to give lateral stiffness and to prevent overturning by the specified lateral forces, and in no case shall be less than one-twentieth of the span.

7.—Depth Ratios.—The depth of trusses preferably shall be not less than one-tenth of the span. The depth of plate girders preferably shall be not less than one-twelfth of the span. The depth of rolled beams used as girders preferably shall be not less than one-twentieth of the span. If less depths are used, the section must be increased so that the maximum deflection will not be greater than if these limiting ratios had not been exceeded.

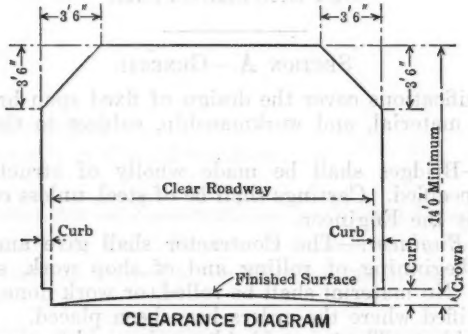
8.—Clearance.—Bridges constructed for highway traffic only shall not encroach on the space indicated by the clearance diagram, Fig. 1. On structures carrying electric railways, the clearance shall be increased to meet the requirements of the case. Clearance from center line of track to the curb on tangent shall not be less than 6 ft. 0 in. and shall be properly increased for curvature and super-elevation of rail.

9.—Traffic Classification of Bridges.—Bridges shall be classified, on the basis of traffic, as follows:

Class A.—City bridges or other bridges carrying a highway traffic of exceptionally heavy load units.
Class B.—Bridges on primary roads.
Class C.—Bridges on secondary roads.
Class D.—Bridges carrying electric railway traffic in addition to highway traffic.

10.—Width of Roadway.—The minimum clear width of roadway shall be 9 ft. for each line of traffic, with a minimum of 12 ft. for one line of traffic.

Bridges designed for two lines of highway traffic preferably shall have a clear width of roadway of not less than 20 ft.



CLEARANCE DIAGRAM

FIG. 1.

11.—Curbs.—The width of curbs shall be not less than 6 in. and preferably shall be not less than 9 in. Their height shall be not less than 9 in.

12.—Dimensions for Calculations.—For the calculation of stresses the length shall be:

For trusses and girders, the distance between centers of bearings;
For floor-beams, the distance between centers of trusses;
For stringers, the distance between centers of floor-beams;

and the depth shall be:

For pin-connected trusses, the distance between centers of pins;
For riveted trusses, the distance between centers of gravity of the chords;
For plate girders, the distance between centers of gravity of the flanges (not to exceed out to out of flange angles) unless section modulus is used.

SECTION 1.—LOADS AND STRESSES

101.—Loads.—Stresses shall be shown separately for the following: Dead load, live load, impact, centrifugal force, and lateral and longitudinal forces. Members shall be proportioned for the combination giving maximum sections, except as otherwise provided.

102.—Dead Load.—In estimating the weight of the structure, for the purpose of computing dead load stresses therein, the following unit weights shall be used:

Steel	490 lb. per cu. ft.
Concrete	150 " " " "
Sand and gravel	120 " " " "
Asphalt-mastic and bituminous macadam	150 " " " "
Granite	170 " " " "
Paving bricks	150 " " " "
Timber	5 " " ft. B. M.
Rails and fastenings	As designed

Live Loads

103.—Roadway Live Load.—The live loads for roadway shall be represented by typical truck loadings. Each typical truck loading shall be considered as occupying one lane of traffic 9 ft. wide. Typical truck loading shall be designated by the letter, H, followed by a numeral indicating the weight in tons of the typical truck loads.

104.—Typical Truck Loadings.—

For Floor System:

H-20.....20-ton trucks.
H-15.....15-ton trucks, or one 20-ton truck.
H-13.....13-ton trucks, or one 15-ton truck.

For Girders and Trusses:

H-20.....600 lb. per lin. ft. and 28 000 lb. concentrated.
H-15.....450 " " " " " 21 000 " "
H-13.....390 " " " " " 18 200 " "

The concentrated loads are to be placed so as to cause maximum effect. Typical trucks shall have total loaded weights distributed as in Fig. 2.

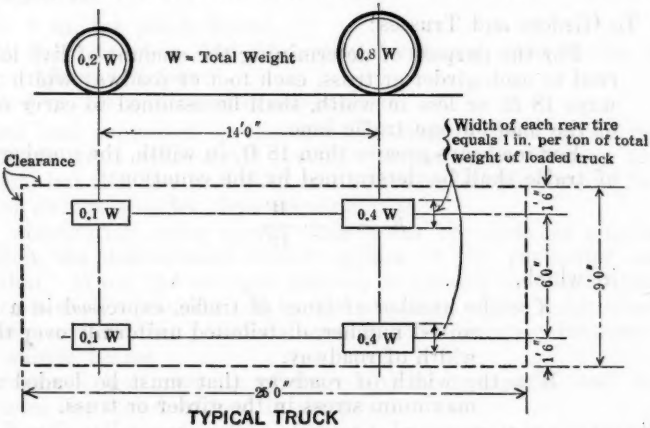


FIG. 2.

105.—Typical Electric Cars.—A train on each track shall be composed of two double-truck cars coupled together, with wheel concentrations and spacing of axles as shown in Fig. 3 or Fig. 4.

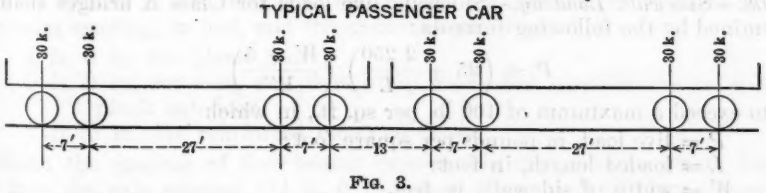


FIG. 3.

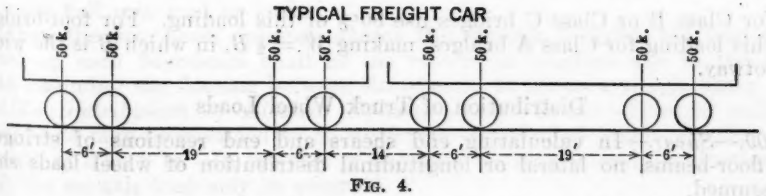


FIG. 4.

106.—Selection of Live Loads.—

Class A Bridges .....H-20 loads.
Class B Bridges .....H-15 loads.
Class C Bridges .....H-13 loads.

107.—Application of Loads.—

To Floor System:

Floor systems shall be designed for as many trucks, side by side, and headed in the same direction, as the width of roadway will accommodate, with a maximum of four. For floors of Class B and Class C bridges the alternate single truck load shall also be considered.

In the design of floor-beams and floor-beam hangers, the following percentages of the resultant live load stresses shall be used:

For one, or two, loaded lanes	100
For three loaded lanes	90
For four loaded lanes	80

To Girders and Trusses:

For the purpose of determining the amount of live load delivered to each girder or truss, each foot of roadway width for roadways 18 ft. or less in width, shall be assumed to carry one-ninth of the load on one traffic lane.

For roadways greater than 18 ft. in width, the number of lanes of traffic shall be determined by the equation:

$$K = 1 + \frac{W}{18}$$

in which,

K = the number of lanes of traffic, expressed in a whole or mixed number, distributed uniformly over the entire width of roadway.

W = the width of roadway that must be loaded to cause maximum stress in the girder or truss.

Class D bridges shall be designed for the highway live loads as herein specified, on any portion of the roadway; or the electric live loads on the car track and the highway live loads on the remaining lanes of traffic. For this condition, the clearance line for electric cars shall be 5 ft. 0 in. from the center line of track.

108.—Sidewalk Loading.—Sidewalk live loads for Class A bridges shall be determined by the following formula:

$$P = \left(25 + \frac{2\,250}{L}\right)\left(\frac{W + 5}{W}\right)$$

not to exceed a maximum of 100 lb. per sq. ft., in which

P = live load, in pounds per square foot;
L = loaded length, in feet;
W = width of sidewalk, in feet.

For Class B or Class C bridges use 80% of this loading. For foot-bridges use this loading for Class A bridges, making $W = \frac{1}{2} B$, in which B is the width of footway.

Distribution of Truck Wheel Loads

109.—Shear.—In calculating end shears and end reactions of stringers and floor-beams, no lateral or longitudinal distribution of wheel loads shall be assumed.

110.—Bending Moment in Stringers.—In determining bending moments in stringers, each wheel load shall be assumed to be concentrated at a point.

When the floor system is designed for one truck, each interior stringer shall be proportioned to support that part of one rear-wheel load, or those

parts of one front-wheel load and one rear-wheel load, represented by a fraction the numerator of which is the stringer spacing, in feet, and the denominator of which is:

4 ft. 0 in. for plank floors;
5 ft. 0 in. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor;
6 ft. 0 in. for reinforced concrete floors.

When the floor system is designed for two trucks, each interior stringer shall be proportioned to support that part of one rear-wheel load, or those parts of one front-wheel load and one rear-wheel load, represented by a fraction the numerator of which is the stringer spacing, in feet, and the denominator of which is:

3 ft. 6 in. for plank floors;
4 ft. 0 in. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor;
4 ft. 6 in. for reinforced concrete floors.

The live load supported by the outside stringers shall be the reaction of a wheel in the most unfavorable position, assuming the flooring to act as a simple beam, but this live load shall in no case be less than would be required for interior stringers under these requirements.

These distribution rules govern only when the stringer spacing is not greater than the denominator which applies to the particular case under consideration. When the stringer spacing is greater than this distance, the stringer loads shall be determined by the reactions of the truck wheels placed in the most unfavorable position, assuming the flooring between stringers to act as simple beams.

The combined load capacity of the stringers in a panel shall be not less than the total live and dead load in the panel.

111.—Bending Moment in Floor-Beams.—In determining bending moments in floor-beams, each wheel load shall be assumed as concentrated at a point.

When stringers are omitted and the floor is supported directly on the floor-beams, the latter shall be proportioned to carry that fraction of one axle load, when the floor system is designed for one truck, or of two axle loads, when the floor system is designed for two trucks, the numerator of which is the floor-beam spacing, in feet, and the denominator of which is:

4 ft. 0 in. for plank floors;
5 ft. 0 in. for 4-in. and 6-in. strip floors and wood blocks on a 4-in. plank sub-floor;
6 ft. 0 in. for reinforced concrete floors.

When the spacing of floor-beams exceeds the denominator given, but is less than the axle spacing (14 ft. 0 in.), each beam shall be proportioned to carry the full axle load or loads.

When the floor-beam spacing exceeds the spacing of axles, the load supported on each floor-beam shall be the maximum reaction due to the axle loads, assuming the flooring between floor-beams to act as a simple beam.

112.—Distribution of Electric Railway Wheel Loads.—Electric railway wheel loads may be assumed to be uniformly distributed longitudinally over a length of 3 ft. 6 in. In the case of ballasted floors, a lateral distribution of 10 ft. for an axle load may be assumed.

113.—Impact.—The dynamic increment of the live load shall be added to the maximum computed live load stresses. For floor-beams and stringers, this increment shall be taken as 30%, and for floor-beam hangers as 60%, of the live load stress, provided these percentages are not less than would be given

by the following formula. For girders and truss members, the increment shall be determined by the formula:

$$I = \frac{S}{3}\left(\frac{2\,000 - l}{1\,600 + 10\,l}\right)$$

in which,

I = impact or dynamic increment to be added to live load stresses;
S = computed maximum live load stress;
l = loaded length of a lane of traffic, in feet, producing the maximum stress in the member. For bridges carrying more than one lane of traffic, the aggregate loaded length of all lanes producing the stress, shall be used.

Impact shall not be added to stresses produced by longitudinal and lateral forces.

114.—Allowance for Increase of Live Load.—For the calculation of stress reversal or counter-stresses, the specified live loads, either uniform or concentrated, shall be increased 100% and, for this load condition, the specified unit stresses shall be increased not more than 50 per cent. The resulting sections shall be not less than would have been required had the allowance for overload not been considered.

115.—Centrifugal Force.—Bridges carrying electric railway traffic on a curve shall be designed to resist a lateral force equal to 10% of the moving railway load without impact allowance. This load shall be assumed to act 4 ft. above the rail.

116.—Lateral Forces.—Spans of 150 ft. and less shall be designed to resist a lateral force of 300 lb. per lin. ft. on the loaded chord and 150 lb. per lin. ft. on the unloaded chord. For spans of more than 150 ft., for each additional 30 ft. of span there shall be added 10 lb. per lin. ft. for the loaded chord and 5 lb. per lin. ft. for the unloaded chord.

All lateral forces are to be considered as moving loads.

117.—Viaduct towers shall be designed for a horizontal wind force of 50 lb. per sq. ft. on one and one-half times the vertical projection of the structure unloaded.

118.—Longitudinal Force.—Provision shall be made for the starting and stopping of electric railway trains, with a coefficient of friction of 15 per cent.

119.—Temperature.—Provision shall be made for movement due to temperature. For thermal stresses in fixed arch bridges provision shall be made for stresses induced by a temperature range of 120° Fahr.

120.—Alternate Stresses.—Members subject to alternate stresses of tension and compression shall be proportioned for the kind of stress requiring the larger section. If the alternate stresses occur in succession during the passage of live load, as in stiff counters, each stress shall be increased by 50% of the smaller.

121.—Combined Stresses.—Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses will not exceed the unit stresses given in Article 201. In members continuous over panel points, three-fourths of the bending stress computed as for simple beams shall be added to the axial stress.

122.—For stresses produced by combination of longitudinal or lateral forces with live load, dead load, impact, and centrifugal force, the unit stresses may be increased 25% over those specified in Article 201. When secondary stresses are included the unit stress may be increased 33⅓ per cent. In no case shall the section be less than that required for dead load, live load, impact, and centrifugal force at the unit stresses specified in Article 201, or less than that required if secondary stresses are not considered.

123.—Secondary Stresses.—Secondary stresses shall be avoided where possible in designing and detailing. In ordinary trusses without sub-paneling, secondary stresses due to distortion need not be considered in any member

the width of which, measured parallel with the plane of flexure, is less than one-tenth its length. All other secondary stresses shall be considered.

SECTION 2.—UNIT STRESSES

201.—In proportioning the several parts of the structure, the following allowable stresses shall be used:

Allowable Stresses for Structural and Rivet Steel:

	Kips* per square inch.
Tension, net section	16.0
Compression (one diameter)	16.0
Compression on columns; gross section:† $\dfrac{16.0}{1 + \dfrac{l^2}{13\,500\,r^2}}$ but not to exceed the value for $\dfrac{l}{r} = 40$,	
Bending in extreme fibers of rolled shapes, built sections and girders, net section	16.0
Bending in extreme fibers of pins	24.0
Shear in plate-girder and I-beam webs, net section	12.0
Shear in pins and shop-driven rivets	12.0
Shear in power-driven field rivets and turned bolts	10.0
Shear in hand-driven rivets and rough bolts	7.5
Bearing on pins, shop-driven rivets, outstanding legs of stiffener angles, and other steel parts in contact	24.0
Bearing on power-driven field rivets and turned bolts	20.0
Bearing on hand-driven rivets and rough bolts	15.0
Bearing on rollers, pounds per linear inch, d = diameter of roller, in inches	0.6 d
Bearing on countersunk rivets; one-half the depth of countersink shall be omitted in calculating bearing area and, in metal less than $\frac{3}{8}$ in. thick, countersunk rivets shall not be assumed to carry stress.	
Bending in cast iron	3.0
Shear in cast iron	3.0
Bearing on cast iron	10.0

in which,

l = length of the member, in inches;
r = least radius of gyration of the member, in inches.

For cast-steel shoes and pedestals, the allowable unit stress for structural steel will apply.

For members composed of steel of greater strength than structural grade, the allowable stresses and the numerator of the column formula, should be increased in proportion to the higher yield point of the stronger steel, provided the yield point is not more than 70% of the ultimate strength. In the column formula, the fractional portion of the denominator should be increased in the same ratio.

* 1 kip = 1 000 lb.

† See Table 9 of Final Report on Specifications for Steel Railway Bridge Superstructure, *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 499.

202.—Allowable Pressure on Masonry.—

	Kips per square inch.
Granite masonry	0.8
Limestone and sandstone (good quality)	0.4
Concrete (1:2:4)	0.6

203.—Compression Flanges of Beams and Girders.—The laterally unsupported length of the compression flanges of beams and girders shall not exceed forty times the flange width. The compressive stress, in pounds per square inch, shall not exceed:

$$16.0 - 0.15\,\frac{l}{b},$$

in which,

l = length, in inches, of unsupported flange, between lateral connections or knee-braces;

b = flange width, in inches;

but not to exceed the value for $\frac{l}{b} = 12$

Limiting Length of Members

204.—Compression Members.—The ratio of unsupported length to the least radius of gyration shall not exceed 120 for main compression members, nor 140 for laterals and sway-bracing. In proportioning the top chords of low trusses, the unsupported length shall be assumed as the distance between the rigid verticals. (See Article 326.)

205.—Tension Members.—For main riveted tension members, the ratio of length to least radius of gyration shall not exceed 200.

Section 3.—Details of Design

301.—Thickness of Metal.—The minimum thickness of structural steel shall be $\frac{1}{6}$ in., except for fillers, railings, and unimportant details. Gussets shall not be less than $\frac{3}{8}$ in. thick. Metal subjected to marked corrosive influence shall be of greater thickness.

302.—Open Details.—Details shall be arranged so as to give free access for inspection and painting. Water-pockets shall be avoided.

303.—Eccentric Connections.—Wherever practicable, connections shall be made symmetrical with the axis of the member, and members shall be connected so that their neutral axes intersect at a point; but if eccentric connections are unavoidable the members shall be proportioned so that the combined fiber stress will not exceed the allowed axial stress.

304.—Compression Members.—Compression members shall be designed so that the metal shall be concentrated as far as feasible in the webs and flanges, and so that the center of gravity of the section may be as near the center line of the member as practicable.

305.—Rigid Members.—Hip verticals and members performing similar functions, and the two end panels of the bottom chords of pin-connected bridges preferably shall be rigid.

306.—Counters and Adjustable Members.—Rigid counters are preferred. Adjustable counters, when used, shall be designed for an initial stress of 10 000 lb., and shall have open turnbuckles. Only one set of diagonals in any panel shall be adjustable.

307.—Net Sections.—Net sections shall be used in all cases in calculating tension members; and, in deducting rivet holes, they shall be taken as $\frac{1}{8}$ in. larger than the nominal diameter of the rivet. In sections having rivets staggered, all rows shall be deducted unless arranged so that the net section along

a zigzag line, taking all distances on the diagonal, at 90% of their value, is greater than the corresponding net section across the plate.

The effective area of single angles in tension shall be assumed as the net area of the connected leg plus 50% of the gross section of the unconnected leg. The effective area of a double angle tension member shall be assumed as 80% of the net section of the member, unless the end details and connections are such that the individual angles are held against bending in both directions, in which case the full net area may be used.

308.—Pin-connected riveted tension members shall have a net section back of the pin-hole, parallel to the axis of the member, of not less than the required net section of the body of the member, and shall have a net section through the pin-hole at least one-third larger than the required section. Riveted tension members shall be stitch-riveted where necessary to make a compact member.

309.—Maximum Length of Rivet.—The total thickness of metal connected by rivets shall not exceed six times the nominal diameter of the rivet to be used.

310.—Rivet Spacing.—Rivets shall be proportioned by their nominal diameter. They shall not be spaced, center to center, closer than 3 diameters. Maximum pitch in the line of stress for members composed of plates and shapes shall not exceed sixteen times the thickness of the thinnest material connected, and shall not exceed 6 in. for $\frac{3}{4}$ in. and $\frac{7}{8}$ in., 7 in. for 1 in., 8 in. for $1\frac{1}{8}$ in., and $9\frac{1}{2}$ in. for $1\frac{1}{4}$ in. For angles connected by more than one gauge line, and with rivets staggered, the maximum pitch in each line may be twice the foregoing values. When two or more plates are used in contact, the lines of rivets nearest their edges shall conform to the foregoing, and the rivets on the interior gauge line shall not be farther apart than twenty-four times the thickness of the thinnest material connected in any direction.

In sides of compression members, consisting of only one plate, the pitch of the rivets in the direction transverse to the line of stress shall not be more than thirty times the thickness of the plate.

For cover-plates on compression members, the pitch of the rivets transverse to the line of stress shall not exceed forty-five times the thickness of the plate, provided that, if this pitch is made greater than forty-five times, up to a permissible maximum of fifty times, the excess over forty-five times shall not be considered as effective in resisting stress.

The pitch of rivets at ends of built compression members shall not be more than 4 diameters, for such a distance as will develop the area of the component parts.

Rivets shall not be spaced closer to sheared edges than $1\frac{3}{4}$ diameters, nor to rolled or planed edges, except in flanges of beams and channels, than $1\frac{1}{2}$ diameters, nor farther from the edge than eight times the thickness of the plate.

311.—Use of Bolts.—Unless specifically authorized, bolted connections shall not be used. Bolts, when authorized, shall be unfinished or turned, as may be specified, and shall meet the requirements of Article 416.

312.—Connections and Splices.—No connection, except for lattice-bars and hand-rails, shall contain less than three rivets. All connections and splices, whether in tension or compression, shall be proportioned to develop the full strength of the members, and no allowance in excess of 50% shall be made for milled ends of compression members. Continuous compression members in riveted structures, such as chords and trestle-posts, shall have faced ends and full contact bearings at the joints. Splices shall be as near the panel points as practicable and, in general, shall be on that side of the panel point which is subjected to the smaller stress.

313.—Extra Rivets in Packing.—Filler-plates through which stress is transmitted, or splice-plates not in direct contact with the parts they connect, shall be extended for extra riveting.

314.—Latticed Members.—All segments of members in compression, connected by latticing, shall have batten-plates at each end. The thickness of such plates shall not be less than one-fiftieth of the distance between the rivets connecting them to the compression member. In no case shall the length of such batten-plates be less than one and one-quarter times the distance between the connecting member. Where intermediate batten-plates are used, their length shall be at least three-quarters the width of the member. In compression members, the last rivet in the batten-plate shall preferably also pass through the end of the adjacent lattice-bar. The distance between the connections of latticing shall be such that the individual members between them, composing the column, shall be relatively stronger than the column as a whole.

315.—Tension members composed of shapes shall have their separate segments connected by batten-plates, or by batten-plates and lattice-bars.

316.—Lattice-Bars.—Single lattice-bars generally shall be inclined at an angle of 60° with the axis of the member, and double lattice-bars at an angle of 45° and riveted at the intersection. Single lattice-bars shall have a thickness of not less than one-fortieth, and double lattice-bars not less than one-sixtieth, of the distance between rivets connecting them to the compression member.

317.—Latticing of compression members shall be proportioned to resist shearing stress normal to the member not less than that calculated by the formula:

$$R = \frac{Pl}{4\,000\,y}$$

in which,

R = normal shearing stress, in pounds;
P = strength of column as a compression member, in pounds;
l = length of column, in inches;
y = distance from neutral axis to extreme fiber, in inches.

In compression members with cover-plate, the cover-plate will be assumed to take one-half the shear.

The diameter of rivets shall not exceed one-third the width of the bar.

318.—Reinforcing Pin-Holes.—Where necessary, pin-holes shall be reinforced by plates. These plates shall be connected so as to distribute the bearing pressure proportionately over the full cross-section with a minimum of eccentricity. At least one full-width plate on each segment shall extend not less than 6 in. beyond the near edge of the batten-plates.

319.—Forked Ends.—Forked ends of compression members should be avoided. Where forked ends are used, a sufficient number of pin-plates shall be provided to give each jaw the full strength of the compression member. At least one of these plates shall extend to the far edge of the batten-plates, and the others not less than 6 in. beyond the near edge of the batten-plates.

320.—Floor System.—Floor-beams preferably shall be at right angles to the trusses or main girders and shall be rigidly connected thereto. Spans with floor systems preferably shall have end floor-beams. When end floor-beams are not used, the end panel stringers shall be secured in correct position by end struts securely connected to the stringers and to the main girders or trusses. The end panel lateral bracing shall be rigidly attached to the main girders, or trusses, and shall also be attached to the end struts.

Steel stringers preferably shall be riveted between the floor-beams with end connections to the floor-beam webs.

Suitable floor expansion joints shall be provided and attached to the steel-work at the expansion ends of all spans and at other points where they may be required. Apron-plates, when used, shall be designed to bridge the joint properly and to prevent the deposit of roadway débris on the masonry.

321.—Design of Bracing.—Lateral, longitudinal, and transverse bracing shall be composed of rigid members, and shall have riveted connections.

In general, bracing shall consist of a double system of diagonal members with transverse compression members.

All intersections of lateral and sway-bracing shall be riveted.

322.—Lateral Bracing.—Top lateral bracing shall be provided in all spans, except half-through bridges which shall have the top chord rigidly fixed in position. Through bridges shall be provided with an upper lateral system having struts at least as deep as the top chord, and with post brackets at the intermediate panel points of sufficient strength to maintain the panel in a vertical position under the specified wind pressure. When head-room permits, an overhead system of sway-bracing shall be used. Portal bracing shall be as deep as the clearance will allow.

Each main panel of deck bridges shall be provided with intermediate sway-bracing of sufficient section to carry one-half the maximum increment due to the moving wind loads.

Bottom lateral bracing shall be provided in all bridges except I-beam spans and deck plate girders of less than 50 ft. span.

In either deck or through bridges, the end sway-bracing shall be proportioned to carry the entire upper lateral stress to the support through the end posts of the truss.

Viaduct towers shall be braced transversely and longitudinally by a system of rigid diagonals with riveted connections. Longitudinal and transverse struts shall be placed at caps and bases and at intermediate panel points. All bracing connections shall be made with gusset-plates. The struts at the base of towers shall be strong enough to slide the movable shoes and provide expansion, at 25% friction, with the truss unloaded.

323.—Half-Through Truss Spans.—The vertical truss members and the floor-beam connections of half-through truss spans shall be proportioned to resist a lateral force, applied at the top chord panel points of the truss, determined by the following equation:

$$R = 150\ (A + P)$$

in which,

R = lateral force in pounds;
A = area of cross-section of chord, in square inches;
P = panel length, in feet.

Preferably outrigger brackets attached to the vertical posts on the outside of the trusses shall not be used.

324.—Plate Girders.—The upper corners of through plate girders, where exposed, shall be neatly rounded to a radius consistent with the size of the flange angles and the vertical height of the girder above the roadway. The flange plate, or a plate of the same width, shall be bent around the curve and continued to the bottom of the girder. In a bridge consisting of two or more spans, only the corners of the extreme ends need be rounded, unless the spans have girders of varying heights, in which case the higher girders shall have their top flanges neatly curved down at the ends to meet the top corners of the girders of the adjacent spans.

The upper flanges of through plate girders shall be rigidly braced laterally, at the panel points of the bridge, by knee-brackets with solid webs, which shall extend out to the clearance line.

Plate girders shall be proportioned by assuming that the flanges are concentrated at their centers of gravity, but in no case beyond the back of flange angles. One-eighth of the gross section of the web may be considered as flange area, provided the web is properly spliced to transmit its bending moment. For unusual sections, the section modulus (net tension) may be used.

When flange plates are used, at least one plate on each flange shall extend the full length of the girder. Any additional flange plates shall extend 1 ft. beyond the theoretical point required, and there shall be a sufficient number of rivets at the end of each plate to transmit its stress value before the theoretical point of the next plate is reached.

325.—Stiffeners.—Stiffener angles shall be placed at end bearings and at points of concentrated loading; these stiffeners shall not be crimped and shall have outstanding legs proportioned for bearing and extending as nearly as practicable to the edge of the flange angles.

Webs shall be stiffened by angles riveted thereto in pairs on opposite sides, with outstanding legs not exceeding sixteen times their thickness, nor less than 2 in. plus one-thirtieth of the depth of the girder. Intermediate stiffeners shall be placed at intervals not exceeding the depth of the web, nor 6 ft.

326.—If the depth of the web between the flange angles or side-plates is less than sixty times the thickness of the web, intermediate stiffeners may be omitted.

327.—End Bearings.—Expansion ends shall be firmly secured against lifting or lateral movement. Fixed bearings shall be firmly anchored. Spans of less than 70 ft. may be arranged to slide upon steel plates with smooth surfaces. Spans of 70 ft. or more shall be provided with rollers or rockers. Neither rollers nor rockers shall be used for expansion bearings at the top of trestle-posts. Spans of 70 ft. or more shall have hinged or pin-bearings at both ends.

328.—Expansion rollers shall not be less than 4 in. in diameter for spans of 100 ft. or less, and this minimum shall be increased not less than 1 in. for each additional 100 ft. and proportionally for intermediate lengths. They shall be connected by substantial side-bars and shall be effectively guided so as to prevent lateral movement, skewing, or creeping. The rollers and bearing-plates shall be protected from dirt and water as far as possible, and the construction shall be such that water shall not be retained and that the roller nests may be inspected and cleaned with the least difficulty. Pin-bearing expansion rockers preferably shall be of steel.

329.—Pedestals and shoes shall be used and be designed to secure rigidity and stability and to distribute the reaction uniformly over the entire bearing area. They shall be made preferably of cast steel or structural steel. The bottom bearing widths shall not exceed the top bearing widths by more than twice the depth of the pedestal and, when involving pin-bearings, this depth shall be measured from the center of the pin. Where built pedestals and shoes are used, the web-plates and the angles connecting them to the base-plates shall not be less than $\frac{5}{8}$ in. thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely.

For spans on an inclined grade and without pin or hinged bearings, the sole plates shall be beveled so that the substructure bridge seats will be level.

330.—Anchor-Bolts.—Anchor-bolts for trusses and girders shall not be less than $1\frac{1}{4}$ in. in diameter and shall extend into the masonry not less than 12 in. Washers shall be used under the nut. Anchor-bolts subjected to tension, as in viaduct towers, shall engage 50% more masonry than is required by the uplift.

331.—Eye-Bar Packing.—Eye-bars shall not be packed out of line with the axis of the member more than $\frac{1}{8}$ in. to 1 ft. Members packed on pins shall be held against side movement.

332.—Camber.—For important structures and those designed to carry electric railways, the length of the truss members shall be such that the camber will equal the deflection produced by the dead load plus full live load without impact. Ordinary structures shall be cambered by increasing the length of the top chord approximately $\frac{3}{8}$ in. for each 10 ft. of horizontal projection.

333.—Railings.—Substantial railings shall be provided along each side of the bridge for the protection of traffic. In general, the railings shall be of two classes, *viz.:*

1.—Railings suitable for use on country bridges which are not subject to general pedestrian traffic.

2.—Railings for the protection of pedestrians on bridges of cities and villages.

The top rail shall be located approximately 3 ft. 6 in. above the floor adjacent to the railing and shall be capable of resisting a horizontal transverse force of 100 lb. per lin. ft.

Railings of the first class may consist of not less than two lines of horizontal rails of approved section. Railings of the second class shall consist of an upper and lower rail connected by a suitable web. The clear distance between the floor and the lower rail shall not exceed 6 in.

Preferably the unsupported length of any rail shall not exceed 8 ft., and all connections to the posts, truss members, etc., shall contain not less than two rivets each. Provisions shall be made for movement due to temperature.

Section 4.—Workmanship

401.—General.—All workmanship shall be equal to the best practice in modern bridge shops. The methods used and the character of the work done shall be such that the strength of the various members, as designed, shall be fully realized.

402.—Straightening.—When necessary, the material shall be straightened by methods that will avoid any excessive local internal stresses. Sharp kinks and bends may be cause for rejection.

403.—Shearing Structural Steel.—All shearing shall be neatly done and material exceeding $\frac{3}{4}$ in. in thickness, shall have $\frac{1}{4}$ in. planed from the sheared edges.

404.—Shearing Alloy Steel.—All shearing shall be neatly done and material exceeding $\frac{1}{2}$ in. in thickness, shall have $\frac{1}{4}$ in. planed from the sheared edges.

405.—Rivet Holes.—Material forming parts of a member composed of more than five thicknesses of metal shall be punched, or drilled, with a tool $\frac{1}{16}$ in. smaller than the nominal size of the rivets, and reamed to full size after the parts are assembled. When the holes are made after the parts are assembled, they shall be drilled full size.

406.—Rivet Holes in Structural Steel.—Material forming parts of a member composed of not more than five thicknesses of structural steel may be punched $\frac{1}{16}$ in. larger than the nominal size of the rivets, whenever the thickness of the metal is equal to or less than $\frac{3}{4}$ in. All metal thicker than $\frac{3}{4}$ in. shall be sub-punched and reamed, or drilled, from the solid.

407.—Rivet Holes in Alloy Steel.—Material $\frac{1}{2}$ in., or less, in thickness forming parts of a member composed of not more than five thicknesses of alloy steel may be punched $\frac{1}{16}$ in. larger than the nominal size of the rivets. Material thicker than $\frac{1}{2}$ in. shall be punched, or drilled, with a tool $\frac{1}{16}$ in. smaller than the nominal size of the rivets, and reamed to full size after the parts are assembled, or drilled from the solid. All metal thicker than the nominal size of the rivets, less $\frac{1}{4}$ in., shall be drilled.

408.—Drifting.—Drifting to enlarge unfair holes will not be allowed.

409.—Riveting.—Rivets shall be heated uniformly to a light cherry red color and shall be driven while hot. The heating of the points of rivets more than the remainder will not be permitted. When ready for driving, they shall be free from slag, scale, and other adhering matter, and, when driven, they shall completely fill the holes. Burned, burred, or otherwise defective rivets shall not be driven.

Loose, burned, badly formed, or otherwise defective rivets shall be cut out. Caulking and re-cupping of rivet heads will not be allowed. In cutting out defective rivets, care shall be taken not to injure the adjacent metal and, if necessary, the rivet shanks shall be removed by drilling.

Shop rivets shall be driven by direct-acting riveters where practicable. The riveting machine shall retain the pressure for a short time after the upsetting is complete.

Pneumatic hammers shall be used for field riveting, except when the use of hand tools is permitted by the Engineer.

410.—Dies.—Punch dies shall not be more than $\frac{3}{32}$ in. larger than the punch.

411.—Web-Plates.—Web-plates of girders which have no cover-plates, shall be flush with the backs of the top flange angles or project above them not more than $\frac{1}{8}$ in.

Web-plates of girders that have cover-plates may be $\frac{1}{2}$ in. less in width than the distance back to back of flange angles. The ends of web-plates at splices shall not be more than $\frac{1}{4}$ in. apart.

412.—Splice Plates and Fillers.—Web-splices, plates, and fillers shall fit within $\frac{1}{8}$ in. of the flange angles.

413.—Web Stiffeners.—Web stiffeners at bearings and points of concentrated loads on the flanges shall be faced and brought to bearing against the flange angles.

414.—Lattice-Bars.—The ends of lattice-bars with single rivets shall be neatly rounded.

415.—Floor-Beams and Stringers.—The main sections of floor-beams and stringers shall be of exact length after riveting, and the end connection angles shall be accurately set to length, and square.

416.—Bolts.—The use of bolts in place of rivets will not be permitted, except for special cases. When bolted connections are authorized, the bolts furnished shall be as specified or directed by the Engineer.

Holes for turned bolts shall be carefully reamed and the bolts turned to a driving fit with the threads entirely outside the holes. The heads and nuts shall be hexagonal. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut and the thread burred.

Rough bolts shall be standard machine bolts with hexagonal heads and nuts. The use of "button-head" bolts will not be permitted.

417.—Abutting.—Abutting joints shall have a close fit. The abutting surfaces of compression joints which depend on contact bearing, shall be accurately faced.

418.—Eye-Bars.—Eye-bars shall be made by a process that will produce a uniform product. Welds will not be allowed. The form of the heads may be determined by the dies at the works where the eye-bars are made if satisfactory to the Engineer, but the manufacturers shall guarantee the bars to break in the body when tested to rupture. The thickness of the head shall not be more than $\frac{1}{8}$ in. greater than that of the bar, except by mutual agreement in special cases.

419.—Pin Clearance.—The diameters of pin-holes shall not exceed those of the pins by more than $\frac{1}{32}$ in.

420.—Pins.—Pins more than 7 in. in diameter shall be forged and annealed. Pins 9 in. or more in diameter shall have a 2-in. longitudinal hole bored through the axis. All pins shall be turned.

421.—Upset Ends.—Bars and rods with screw ends shall be upset to provide a cross-sectional area at the root of the thread which shall exceed the net section of the body of the member by at least 15 per cent.

422.—Steel Castings.—Steel castings shall be free from large and injurious blow-holes and shall be annealed.

423.—Bed-Plates.—Expansion bed-plates shall be planed true and smooth. The finishing cut of the planer tool shall be in the direction of expansion. Cast wall-plates shall be planed on top.

424.—Screw-Threads.—Screw-threads shall be U. S. Standard. The nuts shall fit tight.

425.—Welds.—Welds will be allowed only when authorized by the Engineer and where the unit stress is not more than one-half the ordinary specified working stress.

426.—Pilot Nuts.—Two pilot and two driving nuts shall be furnished for each size of pin.

427.—Field Rivets.—Field rivets shall be furnished in excess of the nominal number required to the amount of 10% plus 10 rivets of each size and length.

428.—Shop Painting.—All surfaces that come in contact or that are closed shall receive one coat of approved paint before being assembled. All steel shall be scraped free of scale and shall receive one coat of approved paint before leaving the shops.

429.—All machined surfaces shall receive a coat of white lead and tallow at the shop.

430.—Facilities for Inspection.—The Inspector shall be allowed access to all necessary parts of the works.

431.—Inspector's Authority.—The Inspector shall have the power to reject materials and workmanship which do not fulfill the requirements of these specifications; but, in case of dispute, the Contractor may appeal to the Engineer, whose decision shall be final.

432.—Rejection.—The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection, if they are found to be defective.

Rejected material and workmanship shall be replaced promptly or made good by the Contractor.

Section 5.—Full-Size Eye-Bar Tests

501.—Material Tests.—When the acceptance of eye-bars depends on the results of full-size tests, the quality of the material, as proved by the specimen tests, shall be determined by the manufacturer.

502.—Number of Full-Size Tests.—The number and size of the bars to be tested shall be specified by the Engineer before the mill order is placed.

503.—Selection of Eye-Bars.—The test bars shall be the same section as the bars that are to be used in the structure, and of the same length, if within the capacity of the testing machine. Test bars, representing bars too long for the testing machine, shall be selected from the full-length bar material after the heads on one end have been formed, and shall then be cut and the second head formed to the greatest length that can be tested.

504.—Requirements for Full-Size Tests.—The requirements for full-size eye-bar tests shall be as follows:

	Structural steel annealed eye-bars.
Yield point, in kips per square inch, minimum....	30.0
Ultimate strength, in kips per square inch, minimum	55.0
Elongation percentage, in 18 ft., minimum.....	12

The elongation shall be measured in the body of the bar including the fracture.

505.—Re-Tests.—When a bar fails to meet the requirements, two additional bars of the same size and from the same mill heat may be tested. If two or three bars tested fail, the bars of that size and mill heat shall be rejected.

506.—Payment for Eye-Bars.—Eye-bars thus tested which meet the requirements of these specifications, shall be paid for by the purchaser at the same unit price as for the structure. Bars which fail to meet the requirements of the specifications, and all bars rejected as a result of the tests, shall be at the expense of the Contractor.

Section 6.—Weighing and Shipping

601.—Weight Paid For.—The payment for pound-price contracts shall be based on the weight of metal in the fabricated structure, including field rivets shipped. The weight of erection bolts, field paint, and all boxes, crates, or other containers used for packing, together with sills, struts, and rods used for supporting members during transportation, shall be excluded.

Weights paid for shall be shop scale weights unless otherwise provided. If specified in the contract, or permitted by the Engineer, computed weights as hereinafter provided may be made the basis of payment.

602.—Variation in Weight.—If the weight of any member is more than 2½% less than the computed weight, it may be cause for rejection. This applies to both pound-price and lump-sum contracts.

If the total scale weight of any structure exceeds the computed weight by more than 2%, the weight in excess of 2% above the computed weight shall not be paid for.

603.—Weighing of Members.—Finished work shall be weighed in the presence of the Inspector, if practicable. The Contractor shall supply satisfactory scales and shall perform all work involved in handling and weighing the various parts.

604.—Computed Weight.—The weight of steel shall be assumed at 490 lb. per cu. ft. The weight of cast iron shall be assumed at 450 lb. per cu. ft.

The weights of rolled shapes, and of plates up to and including 36 in. in width, shall be computed on the basis of their nominal weights and dimensions, as shown on the approved shop drawings, deducting for copes, cuts, and open holes.

The weights of plates wider than 36 in. shall be computed on the basis of their dimensions, as shown on the approved shop drawings, deducting for cuts and open holes. To this shall be added one-half the allowed percentages of over-run in weight given in Article 711.

The weight of heads of shop-driven rivets shall be included in the computed weight, assuming the weights to be as follows:

Diameter of rivet, in inches.	Weight, in pounds, for 100 heads.
½	4.5
⅝	8.5
¾	14.5
⅞	21.5
1	31.0

The weight of castings shall be computed from the dimensions shown on the approved shop drawings, with an addition of 10% for fillets and over-run.

To the total computed weight of metal may be added an allowance of 0.4 of 1% for shop paint.

605.—Marking and Shipping.—Members weighing more than 3 tons shall have the weight marked thereon. Bolts and rivets of one length and diameter, and loose nuts or washers of each size, shall be packed separately. Pins, small parts, and small packages of bolts, rivets, washers, and nuts shall be shipped in boxes, crates, kegs, or barrels, but the gross weight of any package shall not

exceed 300 lb. A list and description of the contained material shall be plainly marked on the outside of each shipping container.

Anchor-bolts, washers, and other anchorage or grillage materials, shall be shipped to suit the requirements of the masonry construction.

606.—Handling Material.—The loading, transportation, unloading, and storing of structural material shall be conducted so that the metal will be kept clean and free from injury.

Section 7.—Structural Steel for Bridges

These specifications conform to the Standard Specifications for Structural Steel for Bridges of the American Society for Testing Materials.

701.—Steel Castings.—The Standard Specifications for Steel Castings (Serial Designation, A27), adopted by the American Society for Testing Materials, shall govern the purchase of steel castings for bridges. Unless otherwise specified, Class B castings, medium grade, shall be used.

I.—Manufacture

702.—Process.—The steel shall be made by the open-hearth process.

II.—Chemical Properties and Tests

703.—Chemical Composition.—The steel shall conform to the following requirements as to chemical composition:

		Structural steel.	Rivet steel.
Phosphorus...	Acid.....	not over 0.06%	not over 0.04 %
	Basic.....	" " 0.04%	" " 0.04 %
Sulphur.....		" " 0.05%	" " 0.045%

704.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus, and sulphur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Article 703.

705.—Check Analyses.—Analyses may be made by the purchaser from finished material representing each melt. The phosphorus and sulphur content thus determined shall not exceed that specified in Article 703 by more than 25 per cent.

III.—Physical Properties and Tests

706.—Tension Tests.—

(*a*).—The material shall conform to the requirements as to tensile properties given in Table 1.

TABLE 1.

Properties considered.	Structural steel.	Rivet steel.
Tensile strength, in pounds per square inch.....	55 000—65 000*	46 000—56 000
Yield point, minimum, per square inch.....	0.5 tensile strength	0.5 tensile strength
	1 500 000†	1 500 000
Elongation in 8 in., minimum percentage.....	Tensile strength	Tensile strength
Elongation in 2 in., " "	22	.....

*See Article 706, Paragraph (*b*).
† See Article 707.

(*b*).—In order to meet the required minimum tensile strength of full-size annealed eye-bars, the purchaser may determine the tensile strength to be obtained in specimen tests; the range shall not exceed 14 000 lb. per sq. in., and the maximum shall not exceed 74 000 lb. per sq. in. The material shall conform to the requirements as to physical properties other than that of tensile strength, specified in Articles 706(*b*), 707(*b*), and 708(*b*).

(*c*).—The yield point shall be determined by the drop of the beam of the testing machine.

707.—Modifications in Elongation.—

(*a*).—For structural steel more than $\frac{3}{4}$ in. in thickness, a deduction from the percentage of elongation in the 8 in. specified in Article 706(*a*), of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above $\frac{3}{4}$ in., to a minimum of 18 per cent.

(*b*).—For structural steel less than $\frac{5}{16}$ in. in thickness, a deduction from the percentage of elongation in the 8 in. specified in Article 706(*a*), of 1.25% shall be made for each decrease of $\frac{1}{32}$ in. in thickness below $\frac{5}{16}$ in.

708.—Bend Tests.—

(*a*).—The test specimen for plates, shapes, and bars, except as specified in Paragraphs (*b*), (*c*), and (*d*), shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, flat on itself; for material more than $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material more than $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*).—The test specimen for eye-bar flats shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; for material more than $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen; and for material more than $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to three times the thickness of the specimen.

(*c*).—The 1-in. by $\frac{1}{2}$-in. test specimen for pins, rollers, and other bars, when prepared as specified in Article 709, shall bend cold through 180° around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(*d*).—The test specimen for rivet steel shall bend cold through 180° flat on itself, without cracking on the outside of the bent portion.

709.—Test Specimens.—

(*a*).—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*).—Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*), and (*f*), shall be of the full thickness or diameter of the material as rolled.

(*c*).—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 5, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three rolled sides.

(*d*).—Tension-test specimens for plates and eye-bar flats more than $1\frac{1}{2}$ in. in thickness, and bend-test specimens for plates more than $1\frac{1}{2}$ in. in thickness, may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.

(*e*).—Test specimens for bars more than $1\frac{1}{2}$ in. in thickness or diameter may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.; or tension-test specimens may conform to the dimensions shown

in Fig. 6, in which case the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by ½ in. in section.

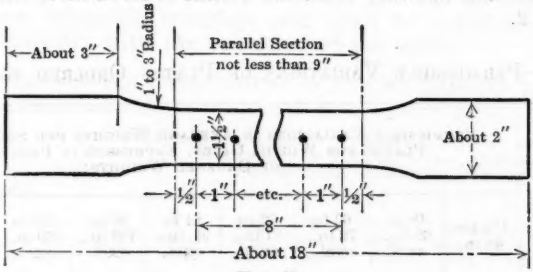


FIG. 5.

(*f*).—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 6. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section.

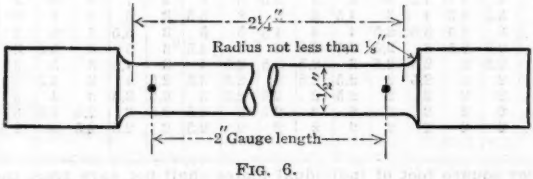


FIG. 6.

(*g*).—The tension-test specimen shown in Fig. 6 and the 1-in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars more than 1½ in. in thickness or diameter, midway between the center and surface.*

(*h*).—The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not more than $\frac{1}{16}$ in.

(*i*).—Test specimens for rivet bars which have been cold drawn, shall be normalized before testing.

710.—Number of Tests.—

(*a*).—One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*).—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*).—If the percentage of elongation of any tension-test specimen is less than that specified in Article 706(*a*), and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight and Thickness

711.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified, except in the case of sheared plates which

* The gauge length, parallel portions, and fillets shall be as shown in Fig. 6, but the ends may be of any form which will fit the holders of the testing machine.

shall be covered by the following permissible variations (1 cu. in. of rolled steel is assumed to weigh 0.2833 lb.):

(*a*).—When ordered to weight per square foot: The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 2.

TABLE 2.—PERMISSIBLE VARIATIONS OF PLATES ORDERED TO WEIGHT.*

Ordered weight, in pounds per square foot.	PERMISSIBLE VARIATIONS IN AVERAGE WEIGHTS PER SQUARE FOOT OF PLATES FOR WIDTHS GIVEN, EXPRESSED IN PERCENTAGES OF ORDERED WEIGHTS:																	
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in., or over.	
	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under
Under 5	5	3	5.5	3	6	3	7	3	....	....	....	....	....	....	....	....	....	....
5 to 7.5, excl	4.5	3	5	3	5.5	3	6	3	....	....	....	....	....	....	....	....	....	....
7.5 " 10, "	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	....	....	....	....
10 " 12.5, "	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3
12.5 " 15, "	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3
15 " 17.5, "	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3
17.5 " 20, "	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3
20 " 25, "	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3
25 " 30, "	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3
30 " 40, "	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3
40, or over	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3

* The weight per square foot of individual plates shall not vary from the ordered weight by more than one and one-third times the amount given in Table 2.

(*b*).—When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. less than that ordered.

The overweight of each lot† in each shipment shall not exceed the amount given in Table 3.

V.—Finish

712.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—Marking

713.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections, when loaded for shipment, shall be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—Inspection and Rejection

714.—Inspection.—The Inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the Inspector,

* The term "lot" applied to Table 2 means all the plates of each group width and group weight.

† The term "lot" applied to Table 3 means all the plates of each group width and group thickness.

free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

TABLE 3.—PERMISSIBLE OVERWEIGHTS OF PLATES ORDERED TO THICKNESS.

Ordered thickness, in inches.	PERMISSIBLE EXCESS IN AVERAGE WEIGHTS PER SQUARE FOOT OF PLATES FOR WIDTHS GIVEN, EXPRESSED IN PERCENTAGES OF NOMINAL WEIGHTS:								
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl.	84 to 96 in., excl.	96 to 108 in., excl.	108 to 120 in., excl.	120 to 132 in., excl.	132 in. or over.
Under 1/8	9	10	12	14	....	....	....	....	....
1/8 to 3/16 excl.	8	9	10	12	....	....	....	....	....
3/16 " 1/4 "	7	8	9	10	12	....	....	....	....
1/4 " 5/16 "	6	7	8	9	10	12	14	16	19
5/16 " 3/8 "	5	6	7	8	9	10	12	14	17
3/8 " 7/16 "	4.5	5	6	7	8	9	10	12	15
7/16 " 1/2 "	4	4.5	5	6	7	8	9	10	13
1/2 " 5/8 "	3.5	4	4.5	5	6	7	8	9	11
5/8 " 3/4 "	3	3.5	4	4.5	5	6	7	8	9
3/4 " 1 "	2.5	3	3.5	4	4.5	5	6	7	8
1, or over	2.5	2.5	3	3.5	4	4.5	5	6	7

715.—Rejection.—

(*a*).—Unless otherwise specified, any rejection based on tests made in accordance with Article 705 shall be reported within five working days from the receipt of samples.

(*b*).—Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

716.—Rehearing.—Samples tested in accordance with Article 705, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

SECTION 8.—STRUCTURAL NICKEL STEEL

These specifications conform to the Standard Specifications for Structural Nickel Steel of the American Society for Testing Materials.

I.—Manufacture

801.—Process.—The steel shall be made by the open-hearth process.

802.—Discard.—A sufficient discard shall be made from each ingot intended for eye-bars to secure freedom from injurious piping and undue segregation.

II.—Chemical Properties and Tests

803.—Chemical Composition.—The steel shall conform to the following requirements as to chemical composition:

		Structural steel.	Rivet steel.
Carbon		not over 0.45%	not over 0.30%
Manganese		" " 0.70%	" " 0.60%
Phosphorus	Acid	" " 0.05%	" " 0.04%
	Basic	" " 0.04%	" " 0.03%
Sulphur		" " 0.05%	" " 0.045%
Nickel		not under 3.25%	not under 3.25%

804.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of the elements specified in Article 803. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Article 803.

805.—Check Analyses.—Analyses may be made by the purchaser from finished material representing each melt. The chemical composition thus determined shall conform to the requirements specified in Article 803.

III.—Physical Properties and Tests

806.—Tension Tests.—

(*a*).—The material shall conform to the requirements as to tensile properties given in Table 4.

TABLE 4.

Properties considered.	Rivet steel.	Plates, shapes, and bars.	Eye-bar flats and rollers,‡ unannealed.	Eye-bar flats* and plus,‡ annealed.
Tensile strength, in pounds per square inch	70 000–80 000	85 000–100 000	95 000–110 000	90 000–105 000
Yield point, minimum, in pounds per square inch	45 000	50 000	55 000	52 000
Elongation in 8 in., minimum, percentage	1 500 000	1 500 000†	1 500 000†	20
	Tensile strength	Tensile strength	Tensile strength	
Elongation in 2 in., minimum, percentage	.....	.....	16	20
Reduction of area, minimum, percentage	40	25	25	35

* Tests of annealed specimens of eye-bar flats shall be made for information only.
† See Article 807.
‡ Elongation shall be measured in 2 in.

(*b*).—The yield point shall be determined by the drop of the beam of the testing machine.

807.—Modifications in Elongation.—For plates, shapes, and unannealed bars of more than 1 in. in thickness, a deduction from the percentage of elongation specified in Article 806(*a*), of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above 1 in., to a minimum of 14 per cent.

808.—Character of Fracture.—All broken tension-test specimens shall show either a silky or a very fine granular fracture, of uniform color, and free from coarse crystals.

809.—Bend Tests.—

(*a*).—The test specimen for plates, shapes, and bars shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material more than $\frac{3}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*).—The test specimen for pins and rollers shall bend cold through 180° around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(*c*).—The test specimen for rivet steel shall bend cold through 180°, flat on itself, without cracking on the outside of the bent portion.

810.—Drift Tests.—Punched rivet holes pitched two diameters from a planed edge shall stand drifting until the diameter is enlarged 50%, without cracking the metal.

811.—Test Specimens.—

(*a*).—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case, the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*).—Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*), and (*f*), shall be of the full thickness or diameter of material as rolled.

(*c*).—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 5, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three rolled sides.

(*d*).—Tension-test specimens for plates and eye-bar flats more than 1½ in. in thickness, and bend-test specimens for plates more than 1½ in. in thickness, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

(*e*).—Test specimens for bars more than 1½ in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.; or tension-test specimens may conform to the dimensions shown in Fig. 6, in which case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by ½ in. in section.

(*f*).—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 6. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section.

(*g*).—The tension-test specimen shown in Fig. 6 and the 1 in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars more than 1½ in. in thickness or diameter, midway between the center and surface.

(*h*).—The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not more than $\frac{1}{16}$ in.

(*i*).—Test specimens for rivet bars which have been cold drawn, shall be normalized before testing.

812.—Number of Tests.—

(*a*).—One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*).—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*).—If the percentage of elongation of any tension-test specimen is less than that specified in Article 806(*a*) and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight and Thickness

813.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations (1 cu. in. of rolled steel is assumed to weigh 0.2833 lb.):

(*a*).—When ordered to weight per square foot: The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 2.

* The term "lot" applied to Table 2 means all the plates of each group width and group weight.

(*b*).—When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot* in each shipment shall not exceed the amount given in Table 3.

V.—Finish

814.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—Marking

815.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections, when loaded for shipment, shall be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—Inspection and Rejection

816.—Inspection.—The Inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the Inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

817.—Rejection.—

(*a*).—Unless otherwise specified, any rejection based on tests made in accordance with Article 805 shall be reported within five working days from the receipt of samples.

(*b*).—Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

818.—Rehearing.—Samples tested in accordance with Article 805, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

VIII.—Full-Size Tests

819.—Tests of Eye-Bars.—

(*a*).—Full-size tests of annealed eye-bars shall conform to the following requirements as to tensile properties:

Tensile strength, in pounds per square inch...	85 000—100 000
Yield point, minimum, in pounds per square inch	48 000
Elongation in 18 ft., minimum, percentage....	10
Reduction of area, " "	30

(*b*).—The yield point shall be determined by the halt of the gauge of the testing machine.

* The term "lot" applied to Table 3 means all the plates of each group width and group thickness.

PROPOSED LOADING FOR HIGHWAY BRIDGES

Discussion*

By Messrs. N. B. Garver and Harold D. Hussey.†

N. B. Garver,‡ Assoc. M. Am. Soc. C. E. (by letter).§—This paper brings out clearly the fact that most highway bridge specifications are not consistent in the matter of loads for floors and trusses. For some time, it has been the writer's practice to design short span trusses for stresses produced by concentrated truck loads, rather than by the usual uniform load.

There are several distinct advantages to be gained by the adoption of a standard loading. At present, there is no uniformity in specifications or practice regarding the load concentrations on the front and rear axles of trucks. The actual concentrations on axles of various makes of trucks differ widely, but it would seem that a typical loading could be agreed on, that would be reasonable and safe for the various classes of bridges. The author's assumption that three-fourths of the load is concentrated on the rear axle, appears to be rational, but the spacing between axles of 15 ft. seems excessive and 13 ft. would be more nearly typical.

Objection will probably be made to the proposed loading, because of the difficulty of its application. This objection is of little importance, because most highway bridges are designed by men who have had special training in this kind of work which is greatly facilitated by the use of tables and diagrams.

Harold D. Hussey,‖ Esq. (by letter).¶—The writer has read with much interest the discussion on his paper. Several of the points raised may profitably be reviewed in conclusion.

Messrs. Waddell** and Steinman†† would confine a loading of this character to spans of less than 300 ft. in length, using a simple uniform load for longer bridges. In view of the small effect of concentrations on trusses of more than 300-ft. span, the stresses produced by the two kinds of loadings would be nearly the same.

Much discussion centered about the width of 9 ft. for each lane of traffic, as proposed, some writers preferring 10 ft. In the Tentative Specifications for Steel Highway Bridges recently proposed by the American Railway Engineering Association and the Special Committee of the Society on Bridge Design and Construction,‡‡ it is stated that the clear width of roadway is to be

* Discussion on the paper by Harold D. Hussey, Esq., continued from November, 1923, *Proceedings*.
† Author's closure.
‡ Bridge Engr., Arkansas Highway Dept., Little Rock, Ark.
§ Received by the Secretary, November 1, 1923.
‖ Asst. Engr., American Bridge Co., New York, N. Y.
¶ Received by the Secretary, January 22, 1924.
** *Proceedings*, Am. Soc. C. E., November, 1923, p. 1901.
†† *Loc. cit.*, p. 1095.
‡‡ *Loc. cit.*, September, 1923, p. 1379.

taken as the distance between curbs, whereas, the clear distance between steelwork is to be 2 ft. greater in the A. R. E. A. specifications, and preferably 1½ ft. greater in the specifications proposed by the Special Committee of the Society. The proposed width of 9 ft. is a clear operating width. The clear operating width on the roadways in the two specifications referred to, however, is more than the distance between curbs. Thus, two lanes of traffic on an 18-ft. roadway would operate in a width of more than 9 ft. each. This fact is recognized in the A. R. E. A. specifications where it is stipulated that a bridge with an 18-ft. roadway shall be assumed to carry two lanes of traffic, while at the same time a width of 10 ft. for each lane is specified.

Mr. Steinman recommends a change in the proposed loadings for different classes of bridges, and suggests three substitute classes, T-20, T-15 and T-10, respectively, in place of the writer's proposed T-20, T-17.5, and T-15 loadings. Further study shows the desirability of a reclassification of loadings, and the writer would now propose the following:

Class A bridges	T-20 loading
Class B bridges	T-15 loading
Class C bridges	T-12.5 loading
For foreign bridges in undeveloped country	T-10 loading

It is believed that a T-12.5 loading is the lightest that can safely be used for Class C bridges. The uniform load following the truck is 50 lb. per sq. ft., which, used in combination with the concentrations, is considered proper for this class of work.

A T-10 loading would be desirable in the case of bridges for export, of which many are now designed for a floor load of a 6-ton wagon and a truss load of 60 to 80 lb. per sq. ft. of floor.

A comparison of these classifications with those of the Tentative Specifications previously mentioned, is shown in Figs. 6, 7, and 8. This indicates a close agreement between the loadings proposed in the Tentative Specifications submitted by the Special Committee of the Society and the T-loadings. The A. R. E. A. loadings are shown to be more severe than the other two for bridges of Class A and Class B, and somewhat less severe for short-span bridges of Class C. Mr. de Charms* has shown that the T-22.5 loading agrees quite closely with the A. R. E. A. 20-ton truck loading.

A suggestion of a new type of loading is given by Mr. Moore,† namely, a uniform load of 720 lb. per lin. ft. with a concentrated load of 25 000 lb. to be placed at any point. In a thorough discussion of the loading feature of the Tentative Specifications for Steel Highway Bridge Superstructure submitted by the Special Committee of the Society on Bridge Design and Construction, L. H. Shoemaker, M. Am. Soc. C. E., has proposed‡ a similar loading of 600 lb. per lin. ft. with a concentrated load of 26 000 lb.; he shows by diagrams and a table that this loading gives results which agree closely with those obtained from the loadings already discussed. In the application of this type of loading, actual trucks would be used in the design of the floor system, and

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1907.
† *Loc. cit.*, p. 1910.
‡ *Loc. cit.*, December, 1923, p. 2073.

the designated loading would be used only in the design of the trusses and girders.

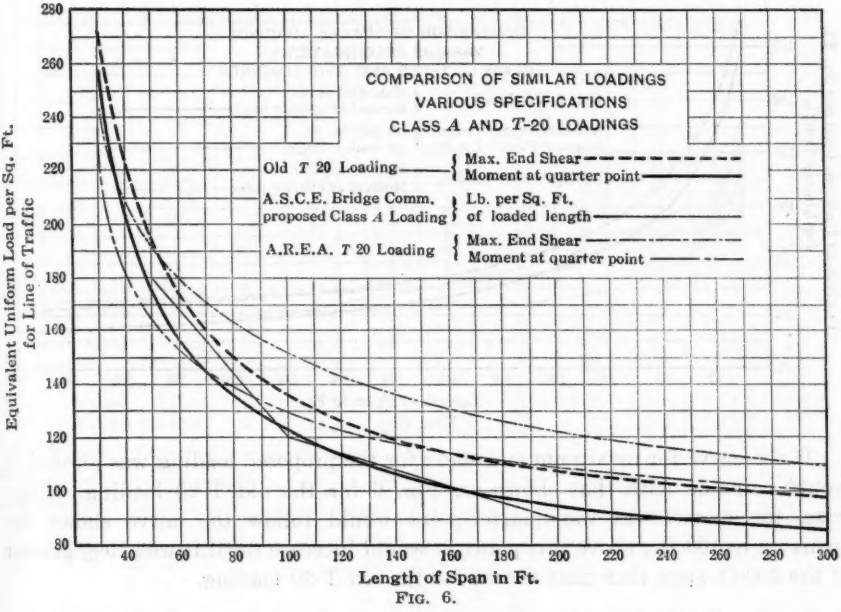


FIG. 6.

It is believed by the writer that a more comprehensive loading would be produced by using the truck concentrations as a nucleus, and placing a uniform load front and rear, as suggested by Mr. Woodruff.* This would make for a

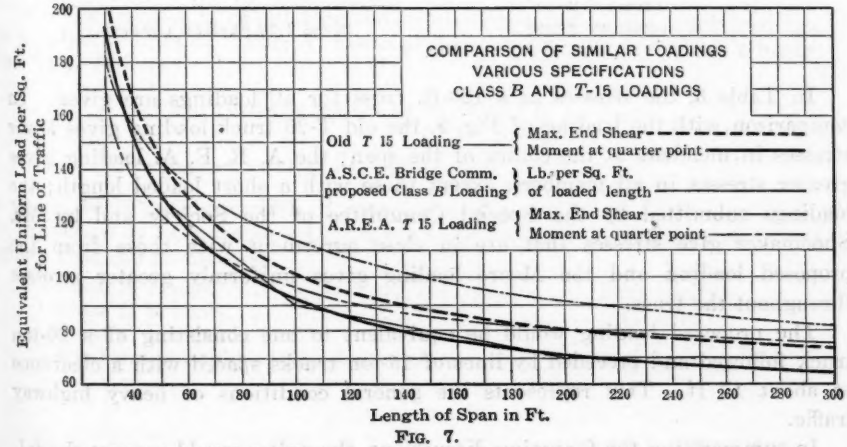


FIG. 7.

simplified specification. The writer, therefore, suggests the loading shown in Fig. 9, as a revision of that shown in his Fig. 2.† It is proposed to use the

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1912.
† *Loc. cit.*, August, 1923, p. 1033.

uniform loads as shown, or to use either portion in combination with the concentrated loads.

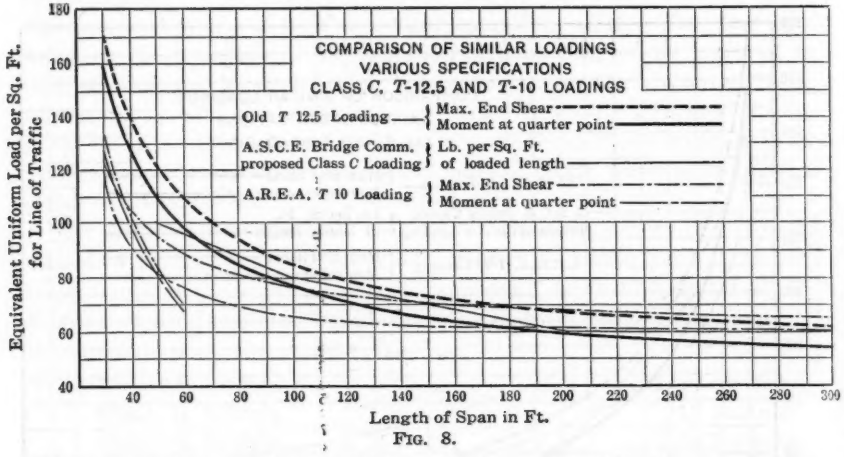


FIG. 8.

If the curve for maximum end shear for the proposed loading was plotted, it would coincide with that shown on Fig. 6 for the old T-20 loading. Likewise, the moment at the quarter-point would follow the curve shown for spans up to 100 ft.; above this point, it would increase until it was 10% greater at the 300-ft. span than that shown for the old T-20 loading.

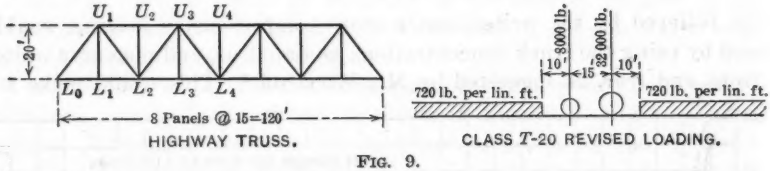


FIG. 9.

In Table 3, the stresses in a 120-ft. truss for all loadings are given. In comparison with the loading of Fig. 9, the old T-20 truck loading gives lower stresses in members at the center of the span; the A. R. E. A. loading gives greater stresses in all members, except those with a short loaded length; the loadings submitted by the Special Committee of the Society and by Mr. Shoemaker give stresses that are in close agreement with those from the proposed loading and the Moore loading gives uniformly greater stresses throughout the truss.

The proposed loading would be equivalent to one consisting of a 20-ton truck followed and preceded by lines of 15-ton trucks spaced with a clearance of about 17 ft. This represents the general conditions of heavy highway traffic.

In summarizing the foregoing discussions, the writer would suggest the following specification:

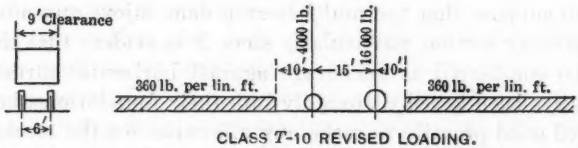
"*Highway Live Loads.*—The highway live load for each line of traffic shall consist of a typical truck followed by, or preceded by, or both followed and preceded by a uniform load. It shall be a multiple of the load shown in

Fig. 10, and shall be assumed to occupy a clearance width of 9 ft. Live loads shall be placed so as to produce the most severe stresses in parts of the structure under consideration.

TABLE 3.—STRESSES IN 120-FT. TRUSS (FIG. 9) FOR DIFFERENT HIGHWAY LOADINGS.

(All Stresses in Kips.)

Member.	T-20 loading.	Old T-20 loading.	Percentage of T-20 loading.	A. R. E. A. loading.	Percentage of T-20 loading.	Am. Soc. C. E. loading.	Percentage of T-20 loading.	Shoemaker loading.	Percentage of T-20 loading.	Moore loading.	Percentage of T-20 loading.
L_0-L_2	+41.3	+41.3	100	+ 46.7	113	+40.4	98	+40.7	99	+ 44.8	108
L_2-L_4	+85.5	+81.0	95	+ 88.6	104	+86.6	101	+87.2	102	+ 96.0	112
U_1-U_3	−69.6	−69.3	100	− 80.6	116	−69.2	99	−69.8	100	− 75.5	108
U_3-U_4	−90.5	−79.5	88	−101.3	112	−92.3	102	−93.0	103	−102.3	113
U_1-L_0	−68.8	−68.8	100	− 78.3	114	−67.3	98	−67.8	99	− 74.6	108
U_1-L_2	+52.8	+52.8	100	+ 59.4	113	+51.7	98	+53.3	101	+ 58.2	110
U_3-L_2	− 39.4	−38.3	97	− 41.9	106	−41.3	105	−40.4	103	− 43.6	111
"	+11.3	+11.3	100	+ 10.2	90	+11.3	100	+11.3	100	+ 11.6	103
U_3-L_4	+28.3	+25.6	90	+ 27.0	95	+30.4	107	+29.2	103	+ 31.0	110
"	−19.0	−17.6	93	− 15.8	83	−19.4	102	−19.4	102	− 20.4	107
Average.....			96	.....	105	.....	101	.....	101	.....	109



CLASS T-10 REVISED LOADING.

FIG. 10.

"For the purpose of calculating the stresses produced by the truck loadings, the number of lines of traffic, K, assumed as symmetrically spaced on a bridge shall be as follows:

"For roadway widths up to 36 ft.: $K = \frac{W}{9}$;

"For roadway widths exceeding 36 ft.: $K = 2 + \frac{W}{18}$;

in which, W is the clear width of roadway, in feet.

"Selection of Live Loads.—Live loads to be used on the different classes of bridges shall be as follows:

"Class A bridges.....T-20 loading
Class B bridges.....T-15 loading
Class C bridges.....T-12.5 loading
For foreign bridges in undeveloped country.....T-10 loading."

IMPROVED TYPE OF MULTIPLE-ARCH DAMS

Discussion*

By Messrs. S. M. Cotten, William P. Creager, and Fred A. Noetzli.†

S. M. Cotten, Esq.‡ (by letter).§—Any practicable design by which the buttresses of a multiple-arch dam may be strengthened as columns and the indeterminate struts now in general use eliminated, is certainly worthy of serious consideration. The design proposed by the author would undoubtedly mark an advance in this direction, and probably the entire accomplishment of the object sought, but at the expense of considerable complication in construction and increase in cost of the buttresses. Evidently, such additional cost would be at least partly offset by the elimination of the struts. It is suggested, however, that approximately the same results could be secured by using deep pilasters at proper intervals, in which case the simplicity of construction would be an advantage, and the final costs no greater, even if more material was required.

With regard to sliding on the foundation, other things being equal, there is no reason to suppose that the multiple-arch dam enjoys any advantage over the gravity masonry section, particularly since it is evident that the arch footings cannot be considered as anchorage against horizontal thrust. Masonry dams have always been keyed thoroughly into their foundations, but it has not been considered good practice to make any allowance for the mechanical bond. If this practice is correct, why would it not also apply to multiple-arch dams? Friction coefficients being based on smooth or relatively smooth contact surfaces, it is evident that a well keyed foundation will develop resistance to sliding considerably in excess of that indicated by the appropriate friction factor, and it would be entirely proper to take such increased resistance into account, provided it could be expressed quantitatively. This difficulty may explain why heretofore "keyed" foundations have not been recognized. If they are to be evaluated now, the expression should be definite and applicable in a general sense, not merely as an attribute of the multiple-arch types.

In analyzing the buttresses for the magnitude and distribution of vertical stress, the author states that either the "free-body buttress" or the "monolith" method may be used, although he gives preference to the latter. In view of the fact that these two methods will give radically different results, even to the extent of differences in sign, it is somewhat difficult to reconcile his indifferenec as to choice. As shown on page 1087,‖ the pressures computed by the two methods, particularly for points toward the up-stream end of the but-

* Discussion on the paper by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., continued from February, 1924, *Proceedings*.

† Author's closure.

‡ Oakland, Calif.

§ Received by the Secretary, December 14, 1923.

‖ *Proceedings*, Am. Soc. C. E., August, 1923.

tress, bear little resemblance. Evidently, if either method is even approximately correct, the other is decidedly wrong, and should not be tolerated. The writer is an advocate of the "monolith" method, as representing a theory both safe and approximately rational, whereas the "free-body buttress" exists only in fancy, and the method, if in error, leads to results that are not on the side of safety. No amount of assumption can change the fact that the arches are actually integral with the buttress and must function with it to the limit of their capacity—a condition evidently unfulfilled in view of the stress magnitude and distribution corresponding in general to this theory as applied to practical cases.

The only plausible argument which can be advanced against the monolith theory is that, compared with the buttress, the arches are so relatively flexible vertically, as to fail to carry their part of the vertical stress. It is undeniable that a part of the arch is relatively flexible, and that if the application of the monolith theory resulted in a high indicated compression under the arch crown, there might properly be some question as to the actual existence of this stress. Fortunately, this is purely an academic question, inasmuch as the analysis of any properly proportioned multiple-arch dam by this method will give such small reactions under the arch ring that there is little doubt as to its ability to absorb them without appreciable deformation; on the other hand, if the dam is inadequately proportioned, the method will indicate a negative reaction, or tension.

Suppose, however, that the arch crown does deflect upward appreciably under the reaction, it must absorb at the same time an appreciable load. There can be no deflection of the arch relative to the buttress at the springing line, consequently, the reaction under the arch and buttress must be approximately equal at points near their junction. Between the crown and the springing line, therefore, the arch must absorb an increasingly greater reaction per unit of area, which even under the least favorable conditions would approximate the results derived under the monolith theory. The latter representing an ideal theory, it is not to be expected that it will fit exactly the actual conditions of stress, any more than that it should fit exactly those in any other of the numerous structures to which it is applied without a question. In case an analysis by this method indicates tension under the arch at the foundation, it is evident that the argument for arch flexibility is entirely void, there being no opportunity for it to accept stress, but that the tendency at least must be for it to take the indicated tension.

Considering an adequately proportioned multiple-arch dam, of what might be termed "standard" design, it is probable that the structure would prove as reasonably safe if analyzed by either of the foregoing methods; but such fact will scarcely justify the use of an erroneous design theory. It is possible, however, to design a multiple-arch dam, which, although just within the limits of safety by the "free-body buttress" method, may be shown to be extremely unsafe by the "monolith" analysis. When the former method indicates compressions that are small on the up-stream side and just within allowable limits on the down-stream side, the latter may indicate up-stream tension and increased down-stream compression, as shown by an example cited later. In

view of the primary importance of these differences, it is scarcely appropriate to adopt an attitude of benevolent neutrality as between the two methods. This matter has been discussed at considerable length by the writer in the technical press.*

The author's conclusions regarding shear combined with compression is well taken in the cases discussed. It is obvious that the effect of the compression is to increase the shearing resistance. How to evaluate this increase is the difficulty. The writer does not believe it is correct to consider that the shear and friction act simultaneously, but it is quite possible that the latter may be taken as a measure of the increase of the former. Lacking the test data that are urgently needed, it would appear reasonable to affirm that the shearing resistance cannot be less than the allowable frictional resistance on the same plane. It is evident that ordinary allowable shearing values do not control in the lower parts of the buttresses, at least, or there would probably be a number of failures to the discredit of this type of dam.

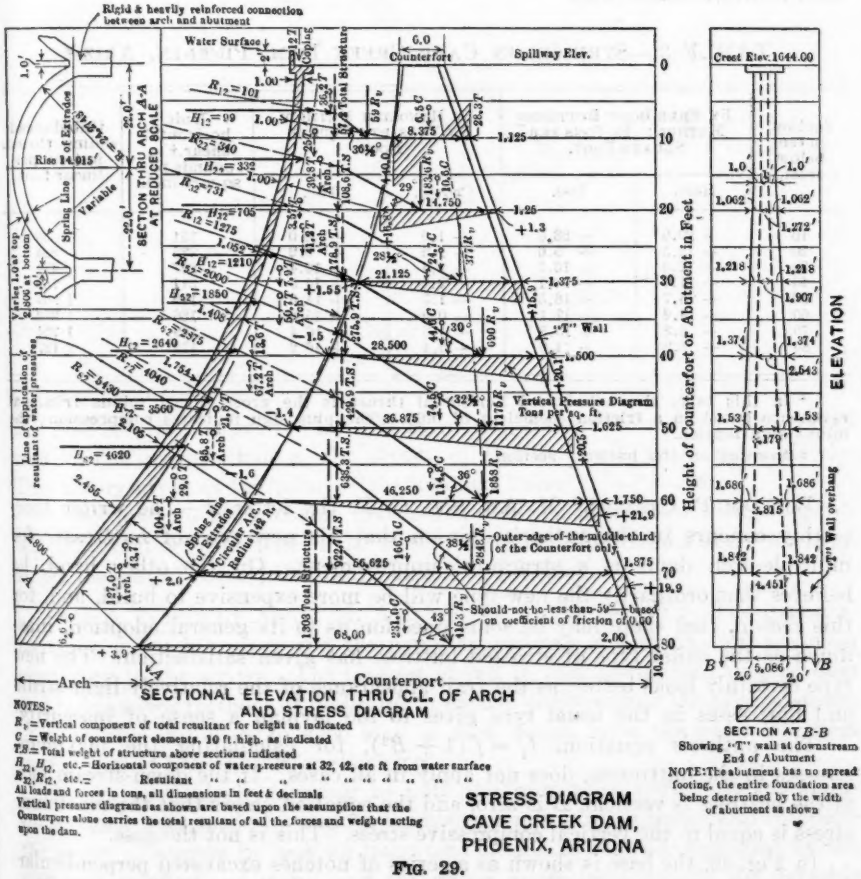
The writer must here take exception to Mr. Noetzli's implication that because the shear in the buttress is combined with compression, it may be disregarded. Having fixed the allowable shearing stress at any section, on some rational grounds depending on the combined compression, it still remains to be seen whether or not this figure is exceeded. Considering, as before, a dam of usual proportions, and taking the frictional resistance on any plane as the measure of the allowable shearing stress, it is probable that the latter will not be greatly exceeded, if at all. In the case of one dam design, however, of which the writer has personal knowledge, the net shear on an unreinforced buttress was 145 lb. per sq. in., after deducting 75% of the vertical load on the section as friction. Would Mr. Noetzli recommend such design?

The author's statement concerning tension in the buttresses is of interest. On the basis of the theory of principal stresses, it would appear that it is quite possible to have tensile stresses of considerable magnitude in the buttresses. The behavior of one dam, in particular, has given strong evidence of this, and another dam analyzed some time ago by the writer showed maximum inclined tensile stresses in the plain concrete of the buttress of approximately 200 lb. per sq. in. It would seem rather necessary to consider this tension in buttress design.

Again, the "factor of safety" of a multiple-arch dam is important. That even the worst type developed to date should fail by over-turning is inconceivable; it is doubtful whether its over-topping to any degree would particularly affect its stability regarding rotation. In the case of a dam of the Ambursen type, it may be readily demonstrated that the structure actually becomes more stable when subjected to considerable over-topping than it is for the designed water level, and the same is probably true for a dam of the multiple-arch type. If a multiple-arch dam fails, it will do so either because of sliding, crushing of the foundation, or over-stressing of the structural members composing it. If safely designed for its normal loads, there is little

* In letters to the editor of *Engineering News-Record*, which appeared in the issues of April 13 and July 13, 1922. The replies of Mr. Noetzli and L. H. Nishkian, Assoc. M. Am. Soc. C. E., to the first of these letters appeared in the same publication under date of June 15, 1922.

possibility of its failure due to either of the first two causes, as indicated previously. Also, if the arches are safely designed for the normal load, the chance of their failure in the lower elevations of the dam is virtually eliminated, due to the fact that the worst overload possible would only increase the stress in these members by a small percentage of that anticipated in the design. It appears, therefore, that a dam of this type which is safely designed for its normal load, can collapse under overload only through the primary failure of the upper arch elements or of some part of the buttress. It is evident that


Rigid & heavily reinforced connection between arch and abutment
Water Surface
Coping
6.0 Counterfort
Spillway Elev.
Crest Elev. 1644.00
SECTION THRU ARCH A-A AT REDUCED SCALE
Rise 14.915'
Spring Line
Variable
Line of Extrados
Line of application of resultant of water pressures
Vertical Pressure Diagram Tons per sq. ft.
"T" Wall
Outer edge of the middle third of the Counterfort only
Should not be less than 59° based on coefficient of friction of 0.60
Height of Counterfort or Abutment in Feet
ELEVATION
Arch
Counterport
SECTIONAL ELEVATION THRU C.L. OF ARCH AND STRESS DIAGRAM
SECTION AT B-B
Showing "T" wall at downstream End of Abutment
NOTE: The abutment has no spread footing, the entire foundation area being determined by the width of abutment as shown
NOTES:
R_v = Vertical component of total resultant for height as indicated
C = Weight of counterfort elements, 10 ft. high, as indicated
T.S. = Total weight of structure above sections indicated
H_{12}, H_{42}, etc. = Horizontal component of water pressure at 32, 42, etc ft from water surface
R_{12}, R_{42}, etc. = Resultant " " " " " " " " " " "
All loads and forces in tons, all dimensions in feet & decimals
Vertical pressure diagrams as shown are based upon the assumption that the Counterport alone carries the total resultant of all the forces and weights acting upon the dam.

STRESS DIAGRAM
CAVE CREEK DAM,
PHOENIX, ARIZONA

FIG. 29.

to have any meaning the "factor of safety" must be based on a number of considerations which will vary in their importance with each individual structure. Both the normal and extreme loads on a multiple-arch dam being capable of computation within close limits, also, the corresponding stresses, the structure is situated much more favorably in this respect than almost any other engineering work. Therefore, eliminating the "factor of ignorance" by proper design and construction, it would seem to follow that the required "factor of safety" could properly be reduced to a low figure.

In order to illustrate several of the points brought out in this discussion, particularly regarding generalizations which may be misleading and even incorrect in particular instances, the writer presents a stress diagram (Fig. 29) pertaining to the design for the Cave Creek Dam, near Phœnix, Ariz., and also some tabulated data (Table 3) in the same connection. The stress diagram and computations were made by the writer, then Assistant Engineer with the City of Phœnix. The design was made by Mr. John S. Eastwood, under whose direction the dam was subsequently constructed, the plans having been modified meanwhile.

TABLE 3.—STRESSES IN CAVE CREEK DAM, PHŒNIX, ARIZ.*

Section, in feet below crest.	By Free-Body Buttress Method: In Tons per Square Foot.		By Monolith Method: In Tons per Square Foot.		Unit horizontal shear,† in pounds per square inch.	Unbalanced sliding thrust, in tons per linear foot.
	Heel.	Toe.	Crown.	Toe.		
10	+ 29.9	− 23.8	− 1.0	+ 3.0	151	66
20	+ 12.5	+ 5.6	− 1.5	+ 9.2	235	224
30	+ 2.3	+ 16.1	− 2.4	+ 12.4	227	474
40	+ 1.0	+ 18.1	− 2.4	+ 14.5	211	769
50	+ 3.7	+ 16.5	− 1.7	+ 14.6	199	1 064
60	+ 4.2	+ 17.1	− 0.9	+ 15.8	186	1 392
70	+ 6.3	+ 15.5	− 0.3	+ 16.4	179	1 754
80	+ 7.9	+ 14.6	+ 0.4	+ 16.2	172	2 127

* In this table the unbalanced horizontal thrust is the gross thrust minus frictional resistance based on a friction coefficient of 0.60. The plus sign indicates compression; the minus sign tension.

† Average on the buttress section.

WILLIAM P. CREAGER,* M. AM. SOC. C. E. (by letter).†—The writer thoroughly concurs in the author's opinion that his new type of buttresses for multiple-arch dams is a structural improvement. On the other hand, he believes that ordinarily the new type will be more expensive to build, and, for this reason, that there may be some question as to its general adoption, inasmuch as the usual type of strutted buttress has given satisfaction. The new type certainly looks better, as the frail appearance of the relatively light struts and buttresses in the usual type gives to the layman a sense of insecurity.

The author's equation, $f_1 = f(1 + B^2)$, for calculating the maximum stresses in the buttresses, does not apply in all cases. If the down-stream face of the buttress is vertical, B is zero, and the equation shows that the maximum stress is equal to the vertical compressive stress. This is not the case.

In Fig. 30, the base is shown as a series of notches excavated perpendicular to the direction of the resultant, R. The total resultant, R, is actually supported on the projected base, L'.

The vertical compressive stresses are computed from the vertical component of R, which is a lesser force, and from the actual base, L, which is a larger area; consequently, even with a vertical down-stream face, the vertical compressive stresses are not the maximum stresses in the masonry.

* Vice-Pres., and Chf. Engr., Northern New York Utilities, Inc., and the Power Corporation of New York, Watertown, N. Y.

† Received by the Secretary, January 15, 1924.

The author's equation, which was developed by Cain from Enger's theory, is not generally applicable, because the whole argument is based solely on a triangular dam, with a vertical up-stream face, having a water-line level with the top. The author has found that the theory will not apply to any other shape of dam or loading; and that, for cases where the angle, θ, of the resultant, R, is greater than ϕ, the angle of the down-stream face with the vertical, then f_1

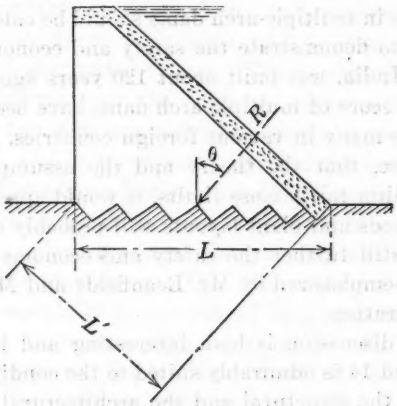


FIG. 30.

will give results that are too low. It is, indeed, rational to assume that, for all values of θ greater than ϕ, the theory of Bouvier* will apply more closely. This theory gives as the value of the inclined stress, $f_1 = f \sec^2 \theta$.

Both equations are probably inaccurate, due to a lack of knowledge regarding the actual deformations of the masonry and the foundations under load; an ample margin of safety, therefore, should be provided.

FRED A. NOETZLI,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The generous reception given his paper and the many expressions of appreciation are most gratifying to the writer. The interest shown in this economical type of dam and in improved methods of designing it clearly demonstrates the general desire of hydraulic engineers to keep step in dam design with the great advances made in other branches of engineering.

It was particularly pleasing to note the numerous additional investigations which were made by various members, tending to augment and complete the paper. Progress in such an important subject is best accomplished by the co-operative efforts of many, and the writer appreciates greatly the valuable suggestions made.

The multiple-arch type of dam permits the use of a sufficiently wide range of designing stresses to satisfy any reasonable demand for factor of safety, and still is less expensive than the old gravity type. The regulations of the Italian Government call for a factor of safety of 8 in concrete, which is

* "Calculs de Resistance des Grands Barrages en Maçonnerie," *Annales des Ponts et Chaussees*, August, 1875.

† Cons. Engr., Los Angeles, Calif.

‡ Received by the Secretary, January 3, 1924.

believed to be extremely high, particularly in view of the factor of safety of 2 required for a gravity dam. Nevertheless, it was estimated that the construction of the 240-ft. Tirso Multiple-Arch Dam* effected a saving of about 20 000 000 lire (approximately $1 000 000) over the cost of a gravity dam.

During recent years, a number of investigations relative to the design of arch and multiple-arch dams have been published by the Society, and in various European countries. There seems to be a fair agreement among engineers as to how the stresses in multiple-arch dams should be calculated. There is also sufficient precedent to demonstrate the safety and economy of this type. The Mir Alam Dam, in India, was built about 120 years ago; during the past two decades more than a score of multiple-arch dams have been built in the United States, and almost as many in various foreign countries.

Granted, therefore, that the theory and the assumptions on which it is based, are known within fairly close limits, it would appear that improvements in construction methods and plant layouts will probably offer the best opportunity for increasing still further the safety and economy of structures of this type. This fact was emphasized by Mr. Beanfield† and Mr. Harza,‡ and merits most careful consideration.

Mr. Cochrane's§ discussion is both interesting and instructive. The dam shown on Figs. 13 and 14 is admirably suited to the conditions for which it was designed, both from the structural and the architectural points of view. The design involves a number of novel features; for instance, the wall at the top of the arch barrel, at right angles to the axis of the arch, seems to provide an economical and safe crest. This wall also stiffens the arch barrel at the top and eliminates many of the uncertainties involved in the design of the uppermost arches.

Mr. Cochrane's comments regarding certain details of the writer's design are well taken. The question of combined cantilever and arch action at the base of the arch barrels will be investigated by the Committee on Arch Dam Investigation, in connection with experiments on a full-sized test dam. It is believed that the elastic deformations of a vertically reinforced arch barrel will be sufficient to avoid any serious troubles arising from this process of adjustment at the base. A reinforced concrete member, of course, is capable of much larger deflections, elastically, than one of plain concrete. The writer agrees with Mr. Cochrane that, for the short arches ordinarily used in multiple-arch dams, the adjustment at the base may not be of any particular consequence, as the deflection of the arches under full water pressure is less than 0.1 in.

With reference to spillways in connection with multiple-arch dams, the writer is in full accord with Mr. Cochrane's remarks on this subject. One of the first multiple-arch dams built in Italy was an overflow dam about 70 ft. in height. The flood waters drop over the arches into stilling pools between the buttresses, thus destroying their force harmlessly. This dam was designed and built by the Italian engineer, G. Ganassini.‖ It was completed in 1920 and

* *Engineering News-Record*, May 10, 1923.
† *Proceedings*, Am. Soc. C. E., January, 1924, p. 58.
‡ *Loc. cit.*, February, 1924, p. 218.
§ *Loc. cit.*, October, 1923, p. 1768.
‖ "Diga dello Scottenno", *Elettrotecnica*, September 25, 1920.

has been in successful operation since then. In many cases, it will be feasible to provide spillways over the low arches at the wings of multiple-arch dams. Siphons or gates may be used to advantage in connection with such a type of construction.

Mr. Jakobsen* and Mr. Cotten† seem to think that the double-wall buttress involves rather expensive construction. The writer's reason for proposing this type of buttress for high dams was suggested primarily by the desire to increase the safety of such dams, and to eliminate the uncertainty involved in the use of horizontal braces. The superiority as regards safety of the double-wall buttress over the ordinary buttress braced by horizontal struts or counterforts has been emphasized by several members who have taken part in the discussion, and in the writer's opinion the safety of the buttresses would even have justified some additional expense if necessary. However, it is most fortunate that a double-wall buttress is not only much safer than an ordinary buttress, but, considering a dam as a whole, that it is also more economical, particularly for high dams. The elimination of the horizontal struts effects a great saving especially in the case of long-span arches. Furthermore, as pointed out by Mr. Cochrane, with such double-wall piers "there will be fewer arches and fewer piers" for a given length of dam, on account of the hollow space between the buttress walls and the consequent greater spacing between the centers of the buttress piers.

Mr. Harza advances an additional important argument in regard to the question of economy of the double-wall buttress type. He states:

"Hollow spaces and interior surfaces of buttresses are much more cheaply constructed than lateral ribs or other irregularities on the exterior surfaces. Internal cavities can be formed with the roughest and cheapest of lumber left in place, if desirable; whereas, external irregularities, such as ribs, would require relatively expensive form work and careful surface treatment, in addition to being weaker structurally against lateral deflection than the H-section proposed by the author."

Thus, form lumber which has served its purpose for exterior surfaces, could be used ultimately for the interior surfaces and eventually be left in place, as suggested by Mr. Harza.

Finally, for multiple-arch dams in which the curvature of the intrados is to be uniform from top to bottom of the dam, the double-wall type of buttress is the only one which is economical. Incidentally, the up-stream part of such a dam may be built of alternating arches and slabs, the arches spanning between the individual buttress piers, and the slabs forming the up-stream closure between the two walls of each pier. Such an arrangement involves many desirable features, particularly from the standpoint of design. In view of the present state of knowledge, it would appear, therefore, to be not only the safest, but also the most economical type. An estimate of quantities and costs made, for instance, in the manner described by Mr. Waller‡ will easily prove this fact.

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1767.

† See p. 296.

‡ *Proceedings*, Am. Soc. C. E., January, 1924, p. 59.

In reply to Mr. Jakobsen's query as to the danger of buckling of the ordinary solid buttress with counterforts, the writer wishes to refer particularly to the discussions by Professor Cain* and Messrs. Cochrane and Harza. For instance, Mr. Harza writes with regard to the double-wall buttress type:

"It is undoubtedly the most stable form of buttress for a given quantity of concrete, the most economical section of high structure for resisting unequal lateral stresses, and offers possibilities of greatly improved architectural treatment over a thin buttress with its multiplicity of bracing."

The writer is grateful to Professor Cain for his encouraging approval of the double-wall buttress type. His investigation of the statical conditions of the arch barrel is valuable both from the theoretical and the practical points of view. Professor Cain analyzes the stresses in elementary horizontal arches, and compares the results with the calculations made with reference to the elementary normal arches. The writer agrees with Professor Cain that the normal-section method is preferable, as it is the most logical. This is also evident if one considers an arch barrel having a flat slope, say, 30° or less, with the horizontal, such as that mentioned by Mr. Parsons,† in which case the elementary horizontal arches would require quite a peculiar shape in order to conform with actual load conditions.

From recent studies made on this subject, the writer is inclined to believe that the actual transmission of loads in the arch barrel of multiple-arch dams takes place mainly through the combined action of elementary normal, horizontal, and vertical arches. The mathematical analysis of this problem would be extremely complicated and hardly worth the effort, because the normal-section method furnishes fairly reliable results, and also leans on the side of safety, as will be shown.

Fig. 31 shows schematically the side elevation of an arch barrel and the supporting buttress. Three elementary arches are indicated, one in a horizontal, one in a normal, and one in a vertical plane. The water pressure, $d\,W$,

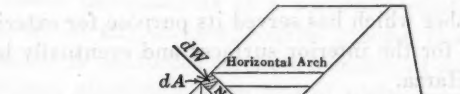


FIG. 31.

acting on the surface element, $d\,A$, is evidently not supported by the normal elementary arch alone, nor by the horizontal or vertical arches alone, but by a combination of all three. Assume, for instance, that the normal arch carries all the pressure. Accordingly, it would be deflected for a certain amount. Inas-

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1777.

† *Loc. cit.*, November, 1923, p. 1924.

much as certain points of the normal arch (in this instance, points at the arch crown) belong also to the horizontal and vertical arches, both these arches must necessarily deflect at identical points for exactly the same amounts as the normal arch. A deflection under load must be due to a certain pressure, and corresponding stresses in all three arches must be approximately in proportion to their relative stiffness, and more exactly according to the law of Nature which makes the total work performed a minimum. It is of importance to note that these three systems stress the concrete of the arch barrel in three different directions, which is an additional and very desirable feature.

In the writer's formulas for stresses in the arch barrel of multiple-arch dams, the elementary normal arches were assumed to support the entire water pressure. It would appear, therefore, that those calculations are much on the side of safety, and that the actual stresses are considerably lower. This should give additional confidence in the safety of the arches of multiple-arch dams; in fact, it would not be possible to crush the elementary normal arches without destroying at the same time the elementary horizontal and vertical arches. It would be difficult, therefore, to imagine any possible condition of over-loading the arches of a multiple-arch dam to the breaking point, and the question of what would happen to the adjacent buttresses in case of failure of one arch seems to be practically eliminated in well designed and well founded structures. In the writer's design of the Horseshoe Dam, and of a number of other similar structures, the arch barrels are of such shape that the horizontal and vertical elementary arches are almost semi-circular in form. The thrust exerted is, therefore, practically parallel to the buttress walls. Doubtless, this adds safety to the construction.

Mr. Parsons properly calls attention to an advantage of the multiple-arch type of dam in that it provides any desired factor of safety against sliding, simply by making the up-stream slope sufficiently flat. In this manner, the weight of the water augments the weight of the concrete, and the sliding factor may be made at little expense even smaller than for a gravity dam. Concrete is expensive material for furnishing weight, and if, for multiple-arch dams, the water in the reservoir can be made to contribute to the stability of the structure, there should be little argument as to its economy.

In case of a yielding foundation, careful consideration must be given to the proper distribution of foundation pressures, in order to avoid uneven settlement of the buttresses. Fortunately, the multiple-arch type of dam with double-wall buttresses is well suited for such a purpose. The two walls of each buttress pier rest on a substantial foundation slab which may be lengthened or widened as required to reduce the pressure on the foundation material to a low value. If the buttresses are made sufficiently long, the resultant of the loads will intersect the base near its center, and the soil pressure will be nearly uniform over the entire base of the dam; the danger from uneven settlements is then reduced to a minimum. For a good rock foundation, the elastic deformations of the bed-rock due to the imposed loads are too small to affect the stress distribution in the arches seriously. Furthermore, inasmuch as the water pressure affects adjacent buttresses simultaneously, and more or less

evenly, the buttresses would evidently also tend to settle evenly, so that the arches would be unaffected statically.

Mr. Gourley's description* of the multiple-arch dam for the Great Lake development in Tasmania is of particular interest for the following reasons:

1.—This dam supersedes a gravity dam, which fact indicates that the engineers of the project clearly recognized the advantages of the multiple arch over the gravity type of dam.

2.—The dam extends 50 ft. below the surface and only 40 ft. above it.

3.—The storage of 1 150 000 acre-ft. is greater than for any other multiple-arch dam on record.

4.—The arches are inclined at an angle of 60° with the horizontal. This is considerably steeper than is customary in the United States.

Mr. Gourley generously endorses the writer's claim for the superiority of the double-wall buttress type, and points out the uncertainties involved in the design of ordinary buttresses with lateral braces.

Mr. Flinn† emphasizes the desirability of making field and laboratory tests on dams in order to verify theoretical investigations. The tests which are being made under the auspices of Engineering Foundation, of which Mr. Flinn is Director, will probably also include experiments on a specially constructed full-sized multiple-arch test dam. Particular attention will be given in these tests to the shearing resistance of the buttresses (ideal maximum stresses) and to the cantilever action at the base of the arch barrels.

Mr. Fisher‡ presents an interesting analysis of the distribution of the stresses in a buttress. The stress diagrams on Fig. 28 show clearly that even with the reservoir full to the crest of the dam, the stresses in the larger part of the buttresses are comparatively low. This would suggest the possibility of using a leaner mixture of concrete, or thinner buttress walls, for those parts of each buttress in which the stresses never reach a value of, say, 300 lb. per sq. in. On the other hand, at small additional cost, a richer mixture of concrete could be used toward the down-stream side of the buttresses where the stresses are more than, say, 400 lb. per sq. in. A similar procedure has been proposed for the Boulder Canyon Dam for which, on account of the height of more than 700 ft., the maximum pressures would probably be between 500 and 600 lb. per sq. in.

Mr. Fisher's reference to the Bear Valley Multiple-Arch Dam, a structure which, although built about 14 years ago, does not show any signs of spalling, should prove that good concrete will not deteriorate even in thin arches (9 in. at the crest) and at the high altitude of 6 500 ft. In the writer's design of the Horseshoe Dam, the arches are 18 in. thick at the crest and about 6 ft. thick at the 200-ft. depth. In addition, the arches will be protected by a layer of gunite, which material is known to be almost impervious even under a head of many hundred feet of water. Consequently, no deterioration of the concrete is anticipated from freezing or thawing.

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* *Proceedings*, Am. Soc. C. E., December, 1923, p. 2067.
† *Loc. cit.*, p. 2068.
‡ *Loc. cit.*, January, 1924, p. 60.

Mr. Schorer's* investigation of the change in the maximum stresses which would result from altering the down-stream slope of the buttresses, and his estimate of the quantity of additional concrete necessary to produce the desired result, is instructive. His findings, that by adding 4.5% to the original yardage of a buttress, the maximum stresses are decreased by about 10%, indicates that any reasonable increase of the factor of safety may be obtained at little additional cost. In some instances, also, it may be of advantage to use a leaner mixture of concrete for the buttresses, and to decrease the unit stresses by lengthening the buttress walls.

Mr. Waller gives a convenient method for estimating the quantities of a multiple-arch dam at any given site. By its use, the determination of the economical height of such a dam becomes possible by estimating the costs of dams for various heights, and finding the cost per acre-foot of reservoir capacity which corresponds to each assumed height. Other conditions remaining the same, the most economical height of dam would evidently correspond to the lowest cost per acre-foot of reservoir capacity.

Mr. Beanfield raises some important questions. He states that,

"In order to obtain the full economic benefit of the recent improvements in multiple-arch dam design, the possibilities of using large size aggregate in the concrete should be studied;" and, further, that "an efficient plant layout is another vital feature in the economical construction of a multiple-arch dam."

The arguments in favor of using large size aggregate are presented so clearly by Mr. Beanfield, profiting by his experience on the Don Pedro Dam, that further comment would seem to be superfluous.

The economy involved in the use of large size aggregate, and the greater shearing and frictional resistance values of the concrete, are features which deserve most careful consideration in future construction. An efficient plant layout, as advocated by Mr. Beanfield, as well as standardized form work, will decrease the cost per cubic yard of concrete, and thus help materially to obtain the full benefit out of this construction.

The writer is grateful to Mr. Harza for his forceful and convincing presentation of the advantages of multiple-arch dams in general and of the double-wall buttress type in particular. Mr. Harza states that,

"In the abstract a multiple-arch dam is an ideal type of structure, and it is hoped that its design will be rapidly developed such that it will be understood sufficiently to gain consideration wherever it is applicable and economically advantageous.

"Concrete is a material primarily adapted to withstand compressive stresses, and in the ideal multiple-arch dam all the material is in compression. In a gravity dam, much of the concrete has no value except weight, and concrete is expensive material for furnishing weight alone. A gravity dam judged from this point of view is, therefore, an uneconomical structure."

In correspondence with the writer, Luigi Luiggi, Hon. M. Am. Soc. C. E., has stated that, in Italy, "the gravity dam is a thing of the past." Another engineer went still further, stating that "the gravity dam is an economic crime." The gravity type of dam has done good service, but engineering science is advancing, and there can be no argument but that the multiple-arch type is

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1926.

the more scientific and economical structure. Mr. Harza also contributes an interesting account of recent activities in multiple-arch dam construction in Italy. The great number of high dams of this type under construction, or contemplated, indicates the marvelously progressive spirit shown by Italian engineers.

The writer agrees with Mr. Harza that there is little merit in the argument as to the danger of injury to the buttresses in case of the collapse of one arch. With the low unit stresses usually assumed, the arches have a factor of safety of at least 4 or 5, and there is no possibility of one arch barrel being overloaded to the breaking point, to say nothing of being wrecked from top to bottom; any argument as to what might happen in such a case is merely speculative.

With regard to the quality of concrete and permissible unit stresses, the writer agrees with Mr. Harza that this question deserves careful consideration. However, concrete as prepared and placed nowadays by the experienced workmen who, fortunately, are numerous, is a material of fairly well known quality and reliability. By good supervision during construction, a product may be secured, which offers a sufficiently large factor of safety to preclude any danger of failure.

Mr. Linden* emphasizes the economical considerations which are so greatly in favor of the multiple-arch type. The fact that the investment will be reduced is of interest both to the owners and the financial institutions which provide the funds. The power consumers, irrigation farmers, consumers of domestic water supply derived from storage reservoirs, the taxpayers, in fact, every one is more or less affected by the amount of money invested in dams, and any progress toward further economy is of general benefit.

Mr. Cotten cites an example of a recently constructed multiple-arch dam which involves certain features that aroused considerable discussion at the time the dam was built. The arch barrels of this dam are curved in such a manner that the slope is about 45° at the base, increasing gradually to a nearly vertical position at the crest. This arrangement would offer certain unquestionable advantages; on the other hand, as pointed out by Mr. Cotten, and as shown by the figures of his Table 3, the dimensions of the buttresses are such that the monolith theory reveals considerable tension stresses at the up-stream side of the "monolith". This theory, although admittedly only approximately correct, is no doubt superior to the old "middle-third" method, and should be used in preference to the latter.

The writer is in accord with Mr. Cotten on the advisability of avoiding all tension stresses in the arch-buttress monolith. He also agrees that no high shearing values should be tolerated if they occur in combination with tension, as the resulting stress would be "diagonal tension". However, no apprehension should be felt for high shear values if they are combined with compression. Theory and tests on various concrete structures agree fairly well on this point.

Mr. Cotten believes that the multiple-arch dam enjoys no advantage over the gravity masonry section as regards sliding, even if the buttresses are

* *Proceedings*, Am. Soc. C. E., January, 1924, p. 61.

well keyed into the rock foundation. The writer wishes to illustrate his point of view by the example of the Horseshoe Dam. According to the drawings and specifications, all over-burden and loose rock are to be removed from the foundation; then, for instance, for the highest buttress, a trench about 22 ft. wide, 220 ft. long, and from 5 to 10 ft. deep, is to be blasted from the solid bed-rock. Both side walls and the base of the trench are to be stepped off, as indicated on the diagrams. Into this rough rock trench, the concrete of the buttress footing is to be cast and carefully worked so as to fill every unevenness both at the bottom and on the side walls. After this is done, the buttress walls are built up monolithically with the base and strongly anchored by using the reinforcement of the buttress walls, which extends into the footing. In a similar manner, all the other buttresses are to be founded individually, but will be connected with each other by the arch foundation which will be between 6 and 10 ft. thick.

In order to produce "sliding" on the foundation of such a buttress, the irregular projections of the bottom and of the side walls of the concrete or of the rock trench would have to be sheared off; the rock abutting against the buttress toe would have to be crushed; and the lowest arches would first have to shear away from their own foundation before they could follow the buttress in such a sliding movement.

Is it not rather far-fetched to speak of "sliding" under such conditions, and to use a sliding factor corresponding to the friction of "loose brick on brick"?

For a gravity dam, the conditions are materially different. The foundation of such a dam is more or less continuous, and the beneficial "keying" effect of the roughened side walls of a large number of individual trenches, as in the case of a multiple-arch dam, is missing. Furthermore, there exists for gravity dams the enormous uncertainty involved in uplift at the base and in horizontal joints, so that, for this type, sliding is really within the realm of possibility. A sliding factor of 0.60 to 0.65 is incidental to the ordinary gravity type, whether required or not, but it is questionable whether the same low standard should be used for other types of dams in which the statical conditions evidently differ materially from those of the ordinary gravity type. In fact, the general adoption of a sliding factor of 0.80 to 0.90, which has been used successfully for some multiple-arch dams, would permit a material saving to be made in future dams of this type. Of course, such a high sliding factor should be used only for dams which are to be founded on solid bed-rock and keyed into the same in a manner similar to that described for the Horseshoe Dam.

Mr. Hall* calls attention to certain restrictions placed on innovations by the State and Federal Government authorities. Government supervision is advisable for such important structures as dams. Most European countries exercise some authority over all such construction. Nevertheless, that rapid progress in these matters may be possible is clearly shown by the splendid advances in dam engineering which have been accomplished during recent years in Italy through the combined efforts of Government and private engineers.

* *Proceedings*, Am. Soc. C. E., February, 1924, p. 222.

With regard to Mr. Hall's suggestion of reducing the angle between construction joints and the resultant of the forces above the joint to one-half the value of the angle of friction, the writer wishes to point out that the direction of the resultant for empty reservoir is vertical, as it is due only to the weight of the dam. Any sliding tendency toward the reservoir side, as intimated by Mr. Hall, seems, therefore, to be rather improbable.

Mr. Creager concurs with the writer that the H-column shape of the improved type of buttresses is structurally far superior to the old type. His question as to economy has been answered previously.

It is believed that the decided increase in stability and safety of a multiple-arch dam with double-wall buttresses will lead to a more general adoption of this type. The comparison relative to cost and safety factor of gravity and multiple-arch dams of the improved type is much in favor of the latter type which deserves, therefore, favorable consideration for all sites where the conditions are suitable for the construction of a masonry dam.

A 5% increase in efficiency of a hydraulic turbine would be considered no doubt a great progress; a saving of 30% to 50% in the cost of a dam should attract no less attention by hydro-electric and irrigation engineers.

Mr. Creager questions the writer's equation, $f_1 = f(1 + B^2)$, for calculating the maximum stresses at the down-stream side of the buttresses. Similar equations were derived by William Cain, M. Am. Soc. C. E.,* and by B. F. Jakobsen, M. Am. Soc. C. E.,† who uses normal sections through the buttresses. For multiple-arch dams which have a comparatively steep up-stream slope and a correspondingly inclined down-stream side of the buttresses, the "normal section" method gives results which are nearly identical with those of the "horizontal section" method. There is, therefore, hardly any doubt as to the correctness of the formulas in question, which were derived independently in three different ways.

In closing, the writer wishes to take this opportunity of recording his gratitude to all those who have so generously enriched the paper by contributing their discussions.

* "Stresses in Masonry Dams", *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 222.
† "Stresses in Multiple-Arch Dams", *Proceedings*, Am. Soc. C. E., August, 1923, p. 1113.

THE RIVER AND HARBOR PROBLEMS OF THE LOWER MISSISSIPPI
A SYMPOSIUM
Discussion*

By Messrs. Hubert Engels, Edward Burr, Charles E. L. B. Davis, and Henry Clay Ripley.†

Dr. Hubert Engels‡ (by letter).§—The scientific and technical value of laboratory work to test river construction can be judged only by a consideration of the results of experiments hitherto made. Although Mr. Freeman has given a general review of what has been attained in this branch of hydraulic engineering, the writer hopes that the following discussion may be of value.

1.—It is to the credit of the French engineer, M. Fargue, to have been the first to make river-model studies.‖ In 1875, the question of improving the navigation route from the sea to Bordeaux was under consideration and there was a difference of opinion about the method to be followed. One side favored an improvement mainly by dredging, whereas, the other argued for a system of river regulation. In order to solve this problem, Fargue made a series of experiments in a model, 60 m. long and 1 m. wide, of that part of the River Garonne in question. These experiments proved that dredging could produce benefit only in combination with regulation works. In this manner, Fargue was able to convince his former opponents of the expediency of his propositions.

2.—The second experiment to solve certain fundamental questions in river regulation with the help of a model was made in 1885. At that time, it was a problem how best to improve the navigable route from Liverpool to Manchester, and it was a question whether this could best be done by means of a canal for deep-water vessels, or by regulation of the Mersey. A special discussion arose over the question of whether certain jetties proposed for the upper reaches of the river would cause sand-banks in the lower reaches, and thereby be harmful to navigation from the ocean to Liverpool. Finally, it was questioned whether the outer bar in its natural position could be deepened by jetties. On this, Professor Osborne-Reynolds took up the subject.¶ For the examination of the questions in dispute, he built small models of the mouth of the Mersey, 1 m. to 2 m. long, made in the form of zinc troughs covered with sand, and proved that in these models all the characteristic dis-

* Continued from January, 1924, *Proceedings*.
† Author's closure.
‡ Professor of Hydr. Eng., Technische Hochschule, Dresden, Germany.
§ Received by the Secretary, December 15, 1923. Translated by Clemens Herschel, Past-President, Am. Soc. C. E.
‖ See International Navigation Congress, Paris, 1892, Part IV, Question 10.
¶ Third International Navigation Congress, Frankfurt-am-Main, 1888, Question 6.

positions of the real estuary repeated themselves. In his report, may be found the following significant statement:

"It may be asking too much of you to believe that the 12-hour rise and fall of the tide in an estuary 30 km. long and 5.5 km. wide is exactly reproduced by an artificial tide lasting 1 min. in a model 2 m. long and 30 cm. wide; and that moreover the effect on the sand bottom of the model will be to move and deposit this sand in the same manner as in Nature and so as to reproduce all the characteristic features of the estuary. But, nevertheless, practically this was accomplished not only in a model 2 m. long, but also in one of a length of only 60 cm. And what is of most importance; the experimental method has proven to be a convenient means of determining in advance the effect of works designed for the improvement of estuaries or of harbors; a method, as inspection has clearly shown, which it would be folly not to apply, before one should begin any notably costly undertaking".

The celebrated engineer, the late Sir L. F. Vernon-Harcourt, continued the experiments of Professor Osborne-Reynolds in 1886. After attaining a normal condition, based on a model time period of about 30 sec., in a model built on a scale of 1 to 500 vertical, and 1 to 30 000 horizontal, he introduced the jetties proposed for the upper reaches of the river. These jetties caused a deposit of sand in the estuary consisting of a long bar that worked toward Liverpool under the effect of the entering flood-tide. After restoring the model to its original condition, jetties were built in the lower part of the estuary, which produced an improvement in the navigable channel through the outer bar. The practical result of these model experiments consisted in the rejection of the jetties proposed for the upper reaches of the Mersey and the creation of a channel along the southern shore.

3.—Immediately after the close of his experiments on the Mersey improvements, Vernon-Harcourt began the study of the improvement of the mouths of the Seine in 1886, for which improvement there were at the time ten projects, all of which contemplated the accomplishment of the desired results by means of jetties, with the aid of dredging. Surveys were available which showed the conditions before the construction of certain jetties in the upper reaches of the river, and others which showed conditions after these regulating works had been constructed, so that Vernon-Harcourt could logically state: If it can be proved by means of the model that the first named state of Nature can be restored in its characteristic forms, and if by means of jetties placed in the model, the same results can be produced that were actually produced by means of the jetties that had been built, then one may be certain that the third and most important step may be taken, namely, that by means of the model, the probable effects of the existing projects for the extension of the jetties toward the mouth of the estuary may be tested. His experiments led to the result that none of the proposed projects would produce the desired result and, in consequence, all these projects were rejected.

4.—As a result of this experiment, the French Government Department in Rouen caused a model of the Seine estuary to be constructed, and from 1890 to 1895 engaged M. Mengin-Lecroeul to make experiments, the results of which were in perfect agreement with those of Vernon-Harcourt.* Con-

* Alb. von Hecke, "La Méthode expérimentale appliquée à l'étude de l'hydraulique fluviale et maritime," Louvain et Paris, 1909.

cerning these experiments, the well known French engineer, the late E. T. Quinette de Rochemont, M. Am. Soc. C. E., stated* that:

"These facts are of a nature to give one confidence in the indications furnished by the experiments made on small models; they lead one to believe that it will often be of advantage to have recourse to experiments of this kind and thus facilitate the solution of difficult problems."

5.—In the first river regulation laboratory built in Dresden, the writer had a reproduction of the natural condition of a river (the Elbe), constructed in the model trough and allowed it to be developed under the action of running water, with the result that the river model that was produced coincided in a remarkable manner with what the actual river showed.† In traveling on a steamer on the Elbe, the writer exhibited a hydrographic survey of his model, without at first telling the river regulation engineers that these plans were based on a model. Only after they had recognized the river district, which was well known to them, did he reveal that the plans shown represented an exact survey of his model.

6.—A notable result was reached in the laboratory for river regulation and shipbuilding in Berlin, with a model of a part of the Weser,‡ by means of which, the model was shown to be in conformity with the actual state of affairs both before and after the regulation of the river. By subsequent experiments on the model, further exemplary improvements on the Weser were indicated, made by mattresses and new constructions on the endangered portions of the river.

7.—By experiments on models in the new river regulation laboratory in Dresden, the proof was furnished that the mouth of a projected inland harbor on the sharp convex right bank of the Rhine down stream from Düsseldorf should be up stream from the middle of the curve, and not down stream from it. The practical result was that the newly projected harbor entrance was adopted.§

8.—The writer deems it proper not to mention at this time the numerous experiments which he made on the action of river regulation works, such as jetties, shore protection works, mattresses, etc. It seems proper, however, to refer briefly to his experiments on the protection of pile foundations against undermining by high water.‖ At the time these experiments were made (about 1894), the generally accepted belief was that such pile foundations were mainly in danger at the down-stream end of the pier, but experiments with small model piers showed that the up-stream end was in danger of undermining. If the piers were not founded sufficiently deep in the sand layer of the model trough, they tipped over toward the current. Investigations were then undertaken on bridge failures that had occurred during high

* Rapports de délégués français au troisième Congrès Internationale de Navigation Intérieure, Paris, 1890.

† H. Engels, "Das Flussbau-Laboratorium der Königl. Tech. Hochschule in Dresden," *Zeitschrift für Bauwesen*, 1900.

‡ Eger, Dix, and Seifert, "Die Versuchsanstalt für Wasserbau und Schiffbau in Berlin", *Zeitschrift für Bauwesen*, 1906 and 1907.

§ H. Engels, "Mitteilungen aus dem Dresdener Flussbau-Laboratorium", *Forschungsarbeiten auf dem Gebiete des Ingenieurwesens*, Heft 200–201, Berlin, 1917.

‖ *Zeitschrift für Bauwesen*, 1894.

water, that confirmed the results of the model experiment. Further experiments on models to discover the means to be taken to protect the model piers had the result, very important in practice, that the generally accepted method of protecting bridge piers by piling rip-rap all around the pier up to the level of low water should be rejected, and that, on the contrary, the up-stream point of the pier should be protected, which protection should extend into the bed of the river rather than project above it.

The economical value of these experiments is shown by the fact that an experiment, costing about $70, enabled a principle to be recognized that saves hundreds of thousands of dollars annually.

The writer began his experiments on the being and behavior of natural watercourses in 1891. He was able to make use of the results found in a report presented to the International Engineering Congress in Chicago, Ill., in 1893.*

From the writer's experiences of the past thirty-two years in experiments on hydraulic structures, he would state that:

A.—The results and achievements of testing river regulation by experiment are encouraging to the end, that this method should continue to be followed and developed.

B.—The cost of pursuing such experiments is small in comparison with the results that may be expected from them.

As a matter of fact it has been possible not only to reproduce on a small scale a certain river district with its principal features but, what is of far greater importance, it is possible to study the changes that may be expected in the river beds in consequence of certain regulation works. One should not expect that the results shown by the model will exactly be reproduced in all particulars in Nature. The experienced and scientifically educated engineer will not expect more than that the kind of change a certain project is to exercise on the river bed may be indicated by the model experiments. The model experiments should only show whether certain general principles on which the proposed works were based have been vindicated. The model will also furnish suggestions about many important questions of construction, the solution of which will give the basis for further computations and projects.

In few countries are the river regulation problems so diverse, so difficult, and of such great economic importance and, at the same time, so urgent, as in the United States. There is no doubt that the erection of a large and comprehensive National hydraulic laboratory in the United States will further the public welfare. A decision thereupon should be facilitated by the votes unanimously passed at certain International Navigation Congresses, as follows:

"According to experiments which were made, more especially by Mr. Vernon-Harcourt, it would be advantageous before the presentation of a jetty project for the mouth of a wide river, with movable bottom, and where the

* H. Engels, "The Limits Attainable in Improving the Navigability of Rivers by Means of Regulation", *Transactions*, Am. Soc. C. E., Vol. XXIX (July 1893), p. 202.

tides introduce silt or sand, to make experiments with a small model, as accurately as possible, showing the results which the several jetty projects produce in the model; not for the purpose of determining the exact shape of the channel and the navigation depths to be expected, but to compare the several projects one with another with regard to the stability of the channel, its size, and the distribution of the deposits."*

"This makes it desirable that hydro-technic laboratories for the study of the behavior of rivers on a small scale should be used to a greater extent and furnished with the necessary means to make experiments upon the different methods proposed for the improvement of navigation; and this should be done as far as possible in connection with the examinations and work upon the rivers themselves."†

EDWARD BURR,‡ M. AM. SOC. C. E. (by letter).§—The author's paper opens for discussion the entire field of control and improvement of large alluvial rivers. The magnitude of the subject is evident from the paper itself. To one with some experience in the control of the Middle Mississippi River no part of the paper carries a stronger appeal than the proposals for thorough liaison between the laboratory and the men striving for practical results on the rivers.

Hitherto, the art of dealing with moving water and shifting sand, on a large scale, has been developed mainly by practical experience. This does not mean that theory and scientific investigation, in the laboratory and in the field, have not played an important part both before and since the time of Humphreys and Abbot and that they should not play an even larger part in the future. The practical, full-scale aspects of the problem are of greater importance and more dependent on field variants of uncertain character than is common in other branches of engineering design and construction.

A simple instance will illustrate the difference between conditions in the laboratory and in the field. The formulas for the flow of water in open channels well serve their purposes and may reasonably be relied on only for conditions that fairly well accord with those on which the deductions were based; but how reliably may they be applied to give the velocity or the quantity of flow in a turbulent stream, irregular in all respects and large enough to have real value to navigation? Field studies of the flow of such streams are made at selected, favorable locations and even then produce results varying materially. Typical turbulent sections are rarely investigated, for evident reasons, and yet it is in such sections or streams that the field engineer finds his most troublesome problems.

Probably greater difficulties will be found in the application of laboratory results to the more intricate problems to be solved in the field, involving the navigation and flood control of large alluvial rivers. Only by the utmost co-operation and appreciation of the field conditions can the laboratory help to solve the construction problems of an intensely practical character.

The great difficulty inherent in these problems is the impossibility of ascertaining with certainty the character, direction, and magnitude of all the factors

* V. Internationaler Binnenschiffahrts-Congress in Paris, 1892. "Summarische Berichterstattung über die Arbeiten des Congresses," Paris, 1892, S. 364.

† XI. Bericht über die Arbeiten des XII. Schiffahrts-Congresses, Philadelphia, 1912, Brüssel.

‡ Col., U. S. A. (*Retired*), New York, N. Y.

§ Received by the Secretary, December 28, 1923.

at work. Engineers after long study and observation have disagreed as to the direction of the prevailing movement of the beach sand at a harbor entrance. A "circular cave" occurs in a bank revetment apparently with no specific cause for the particular incident at that particular point. A 20-in. dredge may work throughout a whole low-water season on a 1 000-ft. crossing, seeking to find the line along which the river will aid in digging the channel instead of promptly refilling it, but without success, although having the benefit of surveys, pilots' advice, and the extensive experience of men familiar with the river.

If the proposed laboratory will furnish any clues to aid in solving such problems, it will repay its costs many times over; if it will help to solve the large general problems, it will be a great National benefit; but it will not fully serve its purpose unless, in addition to increasing existing knowledge of basic principles, it furnishes substantial practical aid to the field engineers in controlling the forces, some of which are obscure or unknown.

The Middle Mississippi, between the Missouri and the Ohio, possesses characteristics that lie between those of the Missouri and the Lower Mississippi in practically all respects touching large questions of navigation and flood control. With less mud and more water, so to speak, than the Missouri and with less water and more mud than the lower river, the middle river has been in a way a field laboratory where methods have been developed for the betterment of navigation, that are satisfactory from technical and economic standpoints. On the Missouri, however, benefits would not be commensurate with equivalent costs and on the lower river the expense of similar methods has been found to be excessive.

The earliest works in the vicinity of St. Louis, built before the Civil War by Captain (later General) R. E. Lee, were of a conventional, solid brush-and-stone type used successfully in some silt-bearing streams until the Seventies. The first dikes in Horse Tail Reach were of this character, but were only partly successful as silt catches and were failures in other respects. Freshets topped and undermined them, ice and drift breached them, and they became a menace instead of an aid. Light, permeable structures—hurdles and dikes—were then tried at Horse Tail and were great successes as silt catches, but the earlier ones were too frail to withstand ice and drift.

Finally, the pile tripod type, accompanied by a continuous mattress to prevent scour along the line of the dike, was successfully developed, and experience was progressively gained in the layout and construction including the location, spacing, and sequence and methods of construction. Longitudinal dikes were entirely discarded. Plans for layout were based on gently leading instead of forcing the current. In most localities the general layout was necessarily based on the tracé of the channel as found at the time, but a layout with gentle curves of suitable radius, avoiding straight reaches and deep bends, was sought for the purpose of stabilizing the location of the low-water channel and securing better depths on the crossings. The merits of a curved layout as compared with one of straight reaches were demonstrated in the 30 miles below St. Louis, including Horse Tail Bar, now only a memory to pilots. Captain,

now Maj.-Gen., O. H. Ernst, U. S. A. (*Retired*), M. Am. Soc. C. E., had much to do with the initial development of this method in the early Eighties.

The middle river has thus been and still is a fruitful field of study in which were progressively developed methods which are technically successful for this section at a reasonable cost. Here, the active forces are at work to full scale—widely different quantities of flow, slopes relatively steep, currents, drift, ice, silt and sand in large quantities, and the usual obscure or unknown factors. The methods heretofore in use may doubtless be improved; some of the successful Missouri methods may be found to be applicable; and new methods, in general or in detail, may be devised by laboratory or other investigations. In any case, however, the experience gained on the middle river is a better point of beginning for the work of the future than the early unsuccessful efforts in Plum Point Reach, or perhaps the more or less detached works on the Missouri, excellent as some of them appear to be in detail.

The interrelationship between slope and depth in such streams is well known; flat slopes are coincident with long, deep reaches, whereas relatively steep slopes mean deep pools with flat slopes separated by shoal crossings with steep pitches, the crossing being in effect a submerged weir of sand.

In its natural condition the middle river had an average slope from St. Louis to the Ohio of about 0.6 ft. per mile, with least low-water depths of 4.5 to 5.0 ft. on crossings such as Horse Tail Bar. Horse Tail Reach after improvement had a least low-water depth of 9.0 to 10.0 ft. and an average slope of about 0.5 ft. In the reach through St. Louis Harbor, where side channels had been closed by dikes and channel widths had been reduced to between 1 500 and 2 000 ft., the slope was about 0.16 ft. and the depths exceeded 20 ft. Kaskaskia Cut-off, the last to occur in the middle river, was narrow and straight for about 6 miles with an average slope of about 0.2 ft. and a depth of 20 ft., but was deteriorating as is customary in such channels.

These details serve to illustrate, as far as a few mere instances may, the relation between slope and depth and the fact that it is practicable, in theory, to create a slope of 0.2 ft. and a depth of 20 ft. as far as St. Louis. They draw attention to the basic principles required to reach such results and to some methods successful in river control. It is conceded to be practicable, theoretically, to straighten the river and to reduce its slope and perhaps to take care of the loss in elevation, as suggested by Mr. Freeman, if the rock bed of the valley is not too far below the level of the alluvial plain. The practical problems arise in holding the river at high water, and at low water as well, to the location and width decided on as necessary to produce and maintain the desired scour. There is preponderant evidence that the river will scour if restrained or modified materially in its regimen. The greatest difficulties of river engineers consist in limiting its scour.

Water will run in a straight line only under ideal conditions; concealed or unappreciated factors cause deflections. Even rain drops on a window pane deviate from a straight line. So with an alluvial river, the unseen causes or undetected impulses as well as those that are easily apparent, give results as typified on the Mississippi River where it long since has been axiomatic that the river will not run in a straight line, either horizontally or vertically.

To bring the river to a predetermined location and width and to keep it there, involves the old problems of river regulation—contraction and bank protection—by the reduction of excessive widths, the protection of existing banks where satisfactory and of new banks when established, and the permanent maintenance of the whole.

Bank protection alone will not suffice; contraction also is needed to secure a general erosion of the river bed to a lower level. Material eroded in one locality moves down stream to re-create like conditions below. A disturbed regimen re-acts both up and down stream, and detached works, however excellent their design and no matter how well they serve their purposes locally, will not be enough. Finally, permanence of works and continued maintenance are essential to control an adversary who never sleeps and is an inexorable inspector of details, especially during his annual periods of over-indulgence in water.

Although the regulation of large alluvial rivers is technically feasible, costs run high for continuous improvement and maintenance following standard methods of construction, if they may be called such, and probably for any other methods that have been suggested. The practical features of Mr. Freeman's vision of a National hydraulic laboratory, may lead to methods for reducing costs by controlling the rivers with works of less extent and of less expense, at the same time of less exposure and more resistance to the destructive forces always in action. Every river engineer should welcome and support these plans —enthusiastically if he be an optimist and, if a doubter, for the purpose of thoroughly testing them at an expense that is small compared to the possible benefits.

CHARLES E. L. B. DAVIS,* M. AM. SOC. C. E. (by letter).†—In the discussion on the problems of the Lower Mississippi River, it is observed that both Mr. Ripley‡ and Professor Haupt§ are still strongly advocating the adoption of a single curved reaction jetty for the mouths of tidal estuaries. To this, there are decided objections.

On the Great Lakes, the numerous dredged channels in the Detroit River, the St. Clair Flats, and the St. Mary's River are straight courses, with widened channels at the angular turns to allow for swinging the ships. This makes all the courses straight lines, easy to navigate and to mark properly by range beacons on shore—lighted at night—enabling vessels to keep in the deep mid-channel. A glance at the map of the Panama Canal will show a similar system—straight courses with angular turns.

In its aids to navigation on the Western rivers the Lighthouse Establishment follows the same plan, namely, stake lights and beacons to mark the straight courses to be followed by vessels.

It has always seemed to the writer that the slight curvature given by the late Capt. Eads to the South Pass jetties was a mistake. The advantage in

* Brig.-Gen., U. S. A. (*Retired*), Atlantic City, N. J.

† Received by the Secretary, January 9, 1924.

‡ "The Economic Location of Jetties," by Henry Clay Ripley, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., August, 1923, p. 1141.

§ *Proceedings*, Am. Soc. C. E., December, 1923, p. 2045.

entering a jettied channel of having a clear unobstructed view of its entire length is so obvious as to require no argument. It far outweighs the benefit claimed for the curved form, namely, that of deflecting the outgoing current slightly in the direction of the littoral current, which is of small velocity in the Gulf of Mexico.

It will be remembered that Capt. Eads submitted two projects to Congress, one for the improvement of Southwest Pass and one for South Pass, Congress selecting the latter apparently because it was much the cheaper. Both projects were based on the report of a Board of Army Engineers which submitted plans and estimates for the improvements. These plans provided for straight jettied channels.

Inasmuch as the improvement of navigable rivers consists largely in removing the kinks from crooked channels, or, in other words, straightening them, it seems to the writer that the deliberate construction of a curved channel is an engineering blunder, one which should never be allowed if it can possibly be avoided.

For two ocean-going ships to pass each other in a restricted waterway having a curved form would be extremely hazardous, particularly in rough weather. It would result in many collisions.

It is a well-known fact that all vessels steer badly in shoal water, and it has also been observed that vessels passing through dredged channels, having little water below their keels, are drawn down ("squat", the sailors call it) and strike bottom. In sending daily notification of the stage of water at the "Soo" to the ore shippers at Duluth, Minn., during the season of navigation, the U. S. Engineer Office, at the Sault Ste. Marie Canal, does not give the channel depth of water as shown by the gauge, but the safe-loading depth for the ore vessels.

In passing near the side of the dredged channel, vessels take a sheer and steer badly. It has also been observed that two vessels passing near each other have a tendency to draw together. On the Lakes, it was found that although the straight channels provided were plainly marked by day-beacons and range-lights and were theoretically of ample width to allow vessels to pass each other, yet collisions were of such frequent occurrence that it became necessary to duplicate all the artificial channels. The Livingston Channel was excavated in the Lower Detroit River for down-bound vessels, the up-bound vessels taking the old channel at the Lime Kiln Crossing; the St. Clair Flats Canal was doubled; and in the St. Mary's River, connecting Lake Superior with Lake Huron, a long straight channel was excavated in solid rock at West Neebish Rapids for down-bound boats, those up-bound using the old channel in the East Neebish.

In the report of the Board of Consulting Engineers for the Panama Canal, the cross-sections of the Canal prisms for the sea-level canal were given, as follows:

"The depth of 40 ft. was adopted as the standard minimum depth in the Canal.

"The standard bottom width in firm earth, including the dredged portions in soft material, between the shore-line of Limon Bay and Bohio, was fixed at 150 ft., the side slopes in the same material being taken at one vertical to two horizontal.

"In rock, the bottom width was taken at 200 ft. with side slopes in the channel of ten vertical to one horizontal, *i. e.*, practically vertical. The side slopes above water, as well as below in firm earth between Bohio and Obispo and south of Paraiso were taken at the inclination of two vertical on three horizontal."

The writer believes that to pass two ocean-going steamships by each other in a rock cut only 200 ft. wide, would be a slow and tedious operation. Had such a canal been constructed, it would have proved entirely inadequate for handling the traffic.

HENRY CLAY RIPLEY,* M. AM. SOC. C. E. (by letter).†—The writer is much gratified at the reception of his paper‡ by the members of the Society who discussed it. Not only have these discussions added much to the value of the paper itself, but they have enabled the writer to bring out many important features concerning the problem, which otherwise might have escaped him.

There are two methods of attacking the question of bar deepening, the deductive and inductive. Mr. Freeman§ regrets that the former method was not used in this paper, whereas the writer is firmly of the opinion that the latter method is the better one in the solution of these problems. Each location has its individual characteristics, and the unknown elements are so great that a deductive solution, the writer considers, is quite impossible.

Some years ago, an eminent engineer of extensive experience, likewise an able mathematician, studied this question by the deductive method. In accordance with his results, he designed plans for the improvement of the entrances to a number of important harbors, including Galveston, Tex., St. John's River, Florida, Cumberland Sound, Georgia and Florida, and Charleston, S. C. Several million dollars were expended in the execution of these works and yet in no single instance did the plan prove to be successful.

The final plan for the improvement of the entrance at Aransas Pass, Texas, was designed by the inductive method. After it was finished in 1909, it developed and maintained the prescribed channel, with a minimum depth of 20 ft., by the natural forces, unaided by dredging or other artificial aids, as was predicted by its designers. Afterward, as has been previously stated, a second jetty was built, and dredging operations were inaugurated to secure a channel depth of 25 ft. However, in 1916, a severe storm in the Gulf completely obliterated this 25-ft. channel, leaving only the original 20-ft. channel along the face of the curved jetty created by the single-jetty plan. It is believed that mathematical computation never could have determined these results.

As regards the cut-and-error method referred to by Mr. Freeman, it may be stated that, by the inductive method, Nature performs this part of the operation and indicates exactly what results may be secured by the proper control of the forces available and the engineer is thus enabled to design a plan for the improvement with the utmost confidence of success.

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* Cons. Engr., Detroit, Mich.

† Received by the Secretary, December 26, 1923.

‡ "The Economic Location of Jetties", *Proceedings*, Am. Soc. C. E., August, 1923, p. 1141.

§ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1624.

Gen. Black* clearly analyzes the conditions governing the formation of bars at the mouths of sediment-bearing streams as being due to the deposit of this sediment when the velocity of the river current is lessened on meeting the open waters of the sea. It is also true, as he states, that this material will continue to be deposited as long as the current flows. It is the province of the engineer to have this deposit take place where it will do the least harm and where the cost of removal, if necessary, will be a minimum and the operation of removal will not obstruct traffic. He further states that:

"The same procedure is necessary when the bar is of drift and wave formation. Here, again, it must be expected that the supply of material will be continued and that periodic artificial removal will be necessary at some future time."

And he quite correctly adds that:

"If the water, after leaving the opening, is guided by a construction concave toward the current, the centrifugal force of the flowing stream will cause the current to follow the guide closely; but this force itself causes a head to be formed against the guide and increases the tendency for the water to seek the lower level at the side. In the concave bank of a stream having a high velocity, this difference of head between the two sides of the stream is measurable, and its effect is seen in the movement of materials eroded from the concave bank across to the low side."

It is interesting to note that during flood stage on the Mississippi River, a difference in elevation on the opposite sides in bends of as much as 1 ft. has been observed where the mean mid-section longitudinal slope was about 0.4 ft. per mile.†

M. R. H. Gockinga has devised a formula for computing the difference in elevation on opposite sides of a stream in bends for varying radii of curvature.‡ His formula is,

$$Y = 0.235\ V^2 \log \left(I + \frac{X}{R} \right) \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, V is the longitudinal velocity, in meters per second, R is the radius of curvature, X and Y are co-ordinates of a point on the surface referred to a system of rectangular axes in a vertical plane through the intersection of the axis of the channel and the water surface.

M. Gockinga found under certain conditions of velocity, slope, width of channel, and radius of curvature, that the elevation of the water surface on the concave side of the channel was such as to indicate a cross-slope on the water surface twice as great as the longitudinal slope.

The actual movement of the material in a curved channel is illustrated in the formula devised by the late Professor Henry Mitchell, M. Am. Soc. C. E., for the cross-profile of the Delaware River near Philadelphia, Pa.§ This formula is, as follows:

$$Y = 33 \left(\frac{1\,100^2 - X^2}{1\,100^2} \right) + \frac{289}{R} \left(\frac{1\,100^2 - X^2}{1\,100^2} \right) 2\,X \ldots\ldots (2)$$

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1792.

† "Hydraulic Principles Governing River and Harbor Constructions", by C. McD. Townsend, p. 19.

‡ *Annales des Ponts et Chaussées*, 1913, p. 112.

§ U. S. Coast and Geodetic Survey Report for 1878, p. 125.

in which, X and Y are vertical co-ordinates of the curve, the origin being at the center of the river where the width is 2 200 ft., and R is the radius of curvature of the center line varying between 16 200 ft. and infinity. The algebraic sign of X must be observed. A computation according to this formula has been made for R equal to 16 200 ft. and for R equal to infinity, as given in Table 4, and the results have been plotted, as shown in Fig. 69. Here, the mean depth is 23 ft., the maximum depth in the straight channel is 33 ft. in the center of the stream, and, in the curved channel, 41 ft. at a point about one-third the width of the stream from the concave side of the channel. This gives an increase in channel depth of 24%, and, inasmuch as the commercial value of a port varies as the cube of the depth of the channel, the increase in commercial value is 92 per cent.*

TABLE 4.—Co-ordinates of River Bottoms in Vertical Planes.
(Compare Fig. 69.)

X.	Y for R = infinity, in feet.	Y for R = 16 200 ft., in feet.
0	33.0	33.0
100	32.7	36.3
200	31.9	38.8
300	30.5	40.4
400	28.6	41.0
500	26.2	40.3
600	23.2	38.2
700	19.6	34.5
800	15.5	29.0
900	10.9	21.5
1 000	5.7	11.9
1 100	0.0	0.0
— 100	32.7	29.2
— 200	31.9	25.0
— 300	30.5	20.6
— 400	28.6	16.3
— 500	26.2	12.0
— 600	23.2	8.2
— 700	19.6	4.7
— 800	15.5	2.1
— 900	10.9	0.3
—1 000	5.7	— 0.5
—1 100	0.0	— 0.0

The increase in depth on the outside of the curve is accompanied by a shoaling on the inside, showing how the scoured material is carried across the channel and deposited on the opposite side. This agrees perfectly with Gen. Black's statement as to the effect of a curved jetty concave toward the current. The question may be asked why such a construction is not the most efficient method of controlling the deposit of the material scoured from a bar channel or that brought down by the waters of a sediment-bearing stream "where it will do the least harm and where the cost of artificial removal, if necessary, will be a minimum and the operation of removal will not obstruct traffic." Here, the material is deposited away from the channel to one side and, if the littoral movement is from the jetty side of the channel, such deposit will not require removal by artificial means.

That the saving thus effected is material is shown by the dredging operations on the Gulf of Mexico. At the South West Pass of the Mississippi River,

* *Engineer and Engineering*, September, 1923, p. 250.

the plan of improvement included converging jetties and dredging, the channel depth to be 35 ft. To June 30, 1922, there had been removed by dredging from 7½ miles of the channel, including the bar, 51 500 000 cu. yd. of material, and the end is not in sight. At Galveston, dredging operations on the outer bar have been more or less continuous since 1895, involving the removal of several million cubic yards of material. In 1909, alone, there was taken from the outer bar 1 366 000 cu. yd. of material; this was eleven years after the completion of the jetties; and in 1922 there was removed by dredging from the outer bar, 2 566 000 cu. yd.*

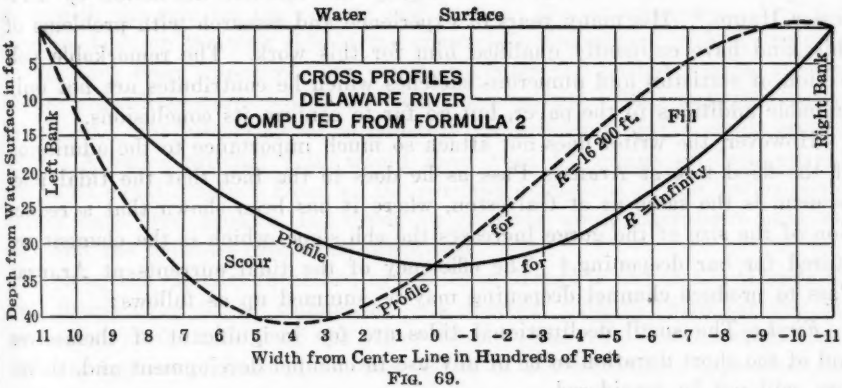

FIG. 69.

It would seem, therefore, as if the statement in Gen. Black's closing paragraph that "the annual accretion to be removed is comparatively small", will need some modification.

His statement that "no works offering a passive resistance or passive guidance, if such terms are permissible, will suffice permanently", is suggestive of the fact that, in the early days of the steamboat, a man actually demonstrated that a steamship could never cross the Atlantic unaided, for she could not carry enough fuel for the trip; and of the humorous poem entitled, "Darius Green and His Flying Machine", in which great fun is made of Darius because he conceived the possibility of a man being able to fly. As late as the time of the Wright Brothers invention, a prominent engineering journal, in a lengthy editorial, tried to discourage any further attempts to improve the flying machine on the ground that it could never become anything but a plaything for a few wealthy people. It is well, therefore, to be a little chary about accepting a statement that a thing can never be done; and especially is this true in view of the results at Aransas Pass with the single curved jetty, where, as already stated, this jetty has secured and maintained a 20-ft. navigable channel along its trace continuously since its completion.

With a strange persistency the channel has maintained itself along the concave face of the curved jetty and has baffled the efforts of those attempting to dislodge it. A second jetty was built on a straight line to the south about 1250 to 1 700 ft. distant, but this failed to budge the channel. Then dredg-

* Report of the Chf. of Engrs., U. S. A., 1922, p. 1047, in which $90 000 is asked for the annual expense of maintenance.

ing operations were undertaken; but the channel continued to hug the curved jetty. Finally, four spur jetties were built from the face of the curved jetty into the channel, thus forcing it away from and out of the influence of the curved structure, thereby inviting deposits. The effect of these spurs will be to put a stop to the further demonstration of the effectiveness of a single curved jetty to maintain a channel without dredging or bar advance and to convert the scheme into a two-jetty plan supplemented by dredging with the inevitable bar advance and continuous expense for maintenance.

The writer cannot commend too highly the valuable discussion by Professor Haupt.* His many years of experience and research with problems of this kind have eminently qualified him for this work. The remarkable collection of statistics and numerous sketches which he contributes are not only valuable additions to the paper, but go far to confirm its conclusions.

However, the writer does not attach so much importance to the admission of the flood tide at Aransas Pass as he does to the fact that the tidal phenomena is the same as at Galveston, where it has been shown that a reduction of the size of the gorge increases the ebb slope, which is the element required for bar deepening.† The efficiency of the tidal currents at Aransas Pass to produce channel deepening may be summed up as follows:

First.—The small declinational tides are too insignificant of themselves and of too short duration to be of any use in channel development and, therefore, will not be considered.

Second.—The great declinational tides, therefore, are the only ones which can furnish sufficient current to produce channel deepening. These tides have a flood and high-water duration of about 18 hours continuously in the 24, which condition enables the tidal basin to fill completely during the flood tide. The long period of high-water slack shows that there is still time to spare after the basin is filled. The short duration of the ebb gives a much greater ebb velocity than flood, and the fact that the low-water slack does not occur at dead low water, but some time after the tide in the Gulf has commenced to rise, shows that the tidal basin does not completely empty itself, having a minimum elevation above the mid-plane between high and low tides.

Third.—The winds have considerable influence over the currents, much more indeed than even the great declinational tides themselves. Onshore winds will sometimes produce almost continuous flood currents for several days, filling the tidal basin to an abnormal height. A shifting of the wind to a "Norther," or an offshore wind, will drive the water out in a few hours, producing strong out-going currents. This result is entirely independent of any moderate reduction of the tidal entrance.

Fourth.—An opening at the shore end of the jetty permits the escape of ebbing water which otherwise would be available for channel development. This opening also admits the entrance of sand at flood-tide and at slack-

* *Proceedings*, Am. Soc. C. E., December, 1923, p. 2045.

† *Transactions*, Am. Soc. C. E., Vol. XXV, p. 543, *et seq.*, under "Tidal Phenomena of Galveston Harbor."

water, whenever there is a littoral movement in either direction. The sand kept in suspension outside by wave action is carried along by the current until it passes in the lee of the jetty where wave action ceases and it is deposited, causing a shoaling of the channel. This deposit either remains as a permanent obstruction or is scoured away by subsequent tidal currents, thus diminishing by so much the force available for channel development.

As regards the conditions at Swinemünde, Professor Haupt has clearly shown that the prevailing littoral drift is from the west and, therefore, the location of the curved jetty on the east side of the channel was not in accordance with the rule for its location laid down in the writer's paper. Had the location been on the west side of the channel, in accordance with the rule, it would have been impossible for vessels to enter the harbor during storms, as the prevailing storm winds are from the northeast. This puts the harbor in the class not susceptible of satisfactory improvement by the single curved jetty. A more rational plan would have been by means of twin curved jetties of equal length. Even so, the periodic extensions of the jetties could hardly have been avoided, but dredging probably would have been unnecessary and the required extensions much less.

Among the sketches furnished by Professor Haupt is one of Rio Grande do Sul, Brazil (Fig. 51*). Comparing this diagram with the plan for the improvement of the mouth of the River Rhone (Fig. 70), a remarkable similarity is apparent. Both plans show jetties of the converging type from the

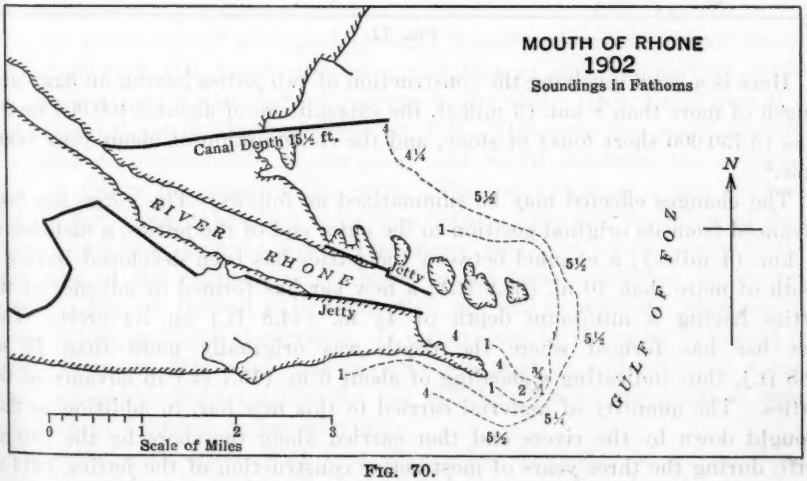

FIG. 70.

shore to the crest of the bar where they become parallel and continue for a short distance beyond. Both jetties were built from the shore out and in both cases the bar advanced as the work progressed, with a slight deepening on the crest. In the case of the Rhone, the work was abandoned, the bar continued to advance and to shoal until the maximum depth on the crest of the

* *Proceedings*, Am. Soc. C. E., December, 1923, p. 2048.

bar was only 6 ft. (See Fig. 70.) The question naturally arises, "Will the bar at Rio Grande behave in a similar manner?" Some light on this matter will be furnished in what follows.

The map of this work, Fig. 71, is taken from the latest survey, of November, 1922, and was not available when the writer's paper was being prepared. This instance shows clearly the necessity of the proper location of jetties in order to secure the best results from their construction.

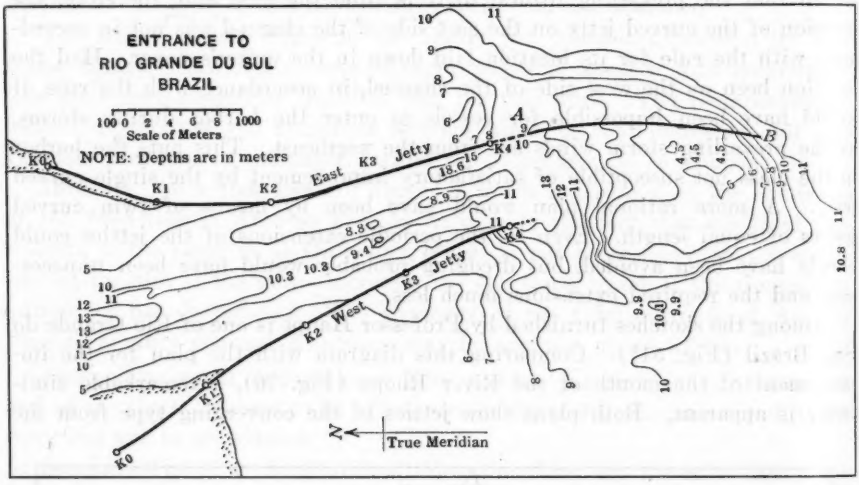

FIG. 71.

Here is a work involving the construction of two jetties having an aggregate length of more than 8 km. (5 miles), the expenditure of about 3 400 000 metric tons (3 750 000 short tons) of stone, and the consumption of about four year's time.*

The changes effected may be summarized as follows: The gorge has been advanced from its original position to the outer end of the jetties, a distance of 6½ km. (4 miles); a channel between the jetties has been developed having a depth of more than 10 m. (32.8 ft.); a new bar has formed in advance of the jetties having a minimum depth of 4½ m. (14.8 ft.) on its crest. This new bar has formed where the depth was originally more than 10 m. 32.8 ft.), thus indicating a shoaling of about 6 m. (19.7 ft.) in advance of the jetties. The quantity of material carried to this new bar, in addition to that brought down by the rivers and that carried along the shore by the littoral drift, during the three years of most active construction of the jetties, 1913 to 1916, was approximately 14 000 000 cu. m. (18 000 000 cu. yd.) or an average of 4 700 000 cu. m. (6 000 000 cu. yd.) per year.†

Undoubtedly, the rate of deposit on the new bar was greater during these first years of channel development than it has been since; but the fact that the bar advanced in the following 6 years at the rate of 112 m. (367 ft.) per year,

* "Travaux du Port et de la Barre de Rio Grande do Sul, Brazil", by M. Basile Malaval, Paris, France.

† *Loc. cit.*, p. 31.

without materially changing the depth on its crest, shows that there is a considerable quantity of material being added to the bar annually, although there is no record of its exact amount.

That this process will continue as time goes on, the volume of the bar increasing while the depth on its crest diminishes (just as at the mouth of the Rhone), there can be no doubt, for the reason that whenever there is an extraordinary discharge the gorge is deepened and the material scoured out is deposited on the crest of the bar. The volume of the excavation thus effected may be enormous. There is shown on Fig. 71 an excavation in the gorge having a maximum depth of 18.6 m. (61.1 ft.). During ordinary conditions, when the current is not so strong, this deep hole is filled more or less with sediment which remains for the next strong outgoing current to wash out again and deposit on the bar, thus repeating the former process. Thus, it will be seen that strong outgoing currents have no tendency to deepen the bar, because their power is exhausted in making an excavation in the gorge and in transporting the material to the crest.

That there is no lack of material for the continual enlargement of the bar is apparent, for, in the past, Canal Norte and the lagoon north of it have been formed by deposits of material furnished by the rivers and by the littoral drift, which has built up a shore to the east of them. No extension of either one or both of the jetties will prevent this process of bar formation, but would only extend or modify the shape of the gorge. However, the conditions here are particularly favorable to the application of the single curved jetty.

It will be seen, by referring to Fig. 71, that the crest of the bar lies to the west of the jetty channel prolonged, showing that the prevailing littoral movement is westward and, hence, the proper location of such a structure would be on the east side of the channel curving to the west. This would give it a location approximately like that shown at *A-B*. This structure would prevent any further deep excavation at the gorge and would constrain the outgoing current to follow the trace of the curve. The excavation of a new channel across the bar would take place along this definite line, and the material thus excavated and that brought down from above would be carried over to the convex side of the channel, as is shown in the cross-profile for the curved channel, in Fig. 69, instead of in front of it, and will be removed subsequently by wave action and littoral currents. If constructed in the order indicated in the writer's paper, this single curved jetty will produce and maintain a permanent navigable channel across the bar sufficient for all the needs of commerce without dredging or bar advance.

The writer quite agrees with Gen. Davis that there may be serious objections to a curvature in a channel of restricted dimensions, such as he cites, but there can hardly be any objection to the gentle curvature given to the suggested curved jetty at the entrance of Rio Grande do Sul, Brazil, where the channel is more than 2 000 ft. wide and where the curvature of the jetty is less than the curvature of the channel in many places in Canal Norte itself. At Aransas Pass, Texas, where the entrance was improved by a single curved jetty, the curvature of the jettied channel is less than the curvature of the channel in the bend immediately inside the entrance; and, in general, the curvature of

a proposed curved jetty will be less than the sharper curves in the natural channel.

In regard to South West Pass of the Mississippi River, it may be said that any curvature to be given to a curved jetty for the improvement of its entrance will be less than the curvature of the channel in many places in the Pass itself. In its official report, the Board of Engineers of 1899 stated that South West Pass, from the Head of the Passes to the Bar, has an ideal channel for navigation. It need not be feared, therefore, that any suitable curved jetty at the entrance of South West Pass will increase in any particular its navigable difficulties.

TENTATIVE SPECIFICATIONS FOR STEEL HIGHWAY BRIDGE SUPERSTRUCTURE

SUBMITTED AS A PROGRESS REPORT OF THE SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION

Discussion*

BY MESSRS. CHARLES EVAN FOWLER, JAMES B. FRENCH, R. W. MACINTYRE, JASPER O. DRAFFIN, AND HENRY W. TROELSCH.

CHARLES EVAN FOWLER,† M. AM. SOC. C. E. (by letter).‡—The proposed specification is in reality a treatise on the design of steel highway bridges and would seem entirely too elaborate for a work-a-day specification. The writer believes that one similar to that published by Thacher twenty-five years ago would be more useful than the symposium presented by the Committee. It may be well to point out again that Sections 4, 5, and 6§ should be published in a separate pamphlet as applying to both highway and railway bridges, in a form agreed on by all the Societies concerned; likewise, Sections 7 and 8.‖ This leaves Sections A, 1, 2, and 3¶ only as relating strictly to highway bridges.

It is not sufficiently clear as to just what class of structures the loads are to be applied. So important a matter cannot be left to the judgment of all users and designers.

The unit stresses are, in the writer's opinion, twenty-five years behind the times, especially in view of the inclusion of traction and impact stresses, which generally have been ignored in designing highway bridges. The unit stresses given cannot be applied to long-span structures; they may not be considered too wasteful for short spans. The inclusion of impact at such large values on all classes of structures imposes a heavy burden on the majority of users, when considered in connection with the low unit stresses permitted, and the actual loads that most highway bridges carry.

This tentative form is too exacting to be used as a general specification, and not broad enough to cover all classes of highway structures, particularly arches, cantilevers, and suspension bridges. However, it may serve a useful purpose as a basis for any special specification.

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* Continued from January, 1924, *Proceedings*. The Final Report of the Special Committee on Specifications for Bridge Design and Construction on Specifications for Steel Highway Bridge Superstructure is also published in this number of *Proceedings*, beginning p. 265.

† Cons. Civ. Engr., New York, N. Y.

‡ Received by the Secretary, January 7, 1924.

§ *Proceedings*, Am. Soc. C. E., September, 1923, pp. 1391, 1393–1394.

‖ *Loc. cit.*, pp. 1395, 1399.

¶ *Loc. cit.*, pp. 1379, 1380, 1384, 1386.

JAMES B. FRENCH,* M. AM. SOC. C. E. (by letter).†—As far as concerns unit stresses, shop workmanship, physical and chemical properties of materials, and, with important exceptions, details of design, there appears little reason why specifications for highway bridges should differ materially from specifications for railway bridges. Although this point of view evidently has been adopted in drafting the specifications for highway bridges, certain differences appear between the latter and those covering railway structures, previously submitted to the Society,‡ which hardly seem justifiable.

For instance, in Section 2, Article 201,§ covering allowable stresses, although the compression on columns is determined by the same formula, the maximum allowable stress is fixed in the highway specifications by limiting the value of $\frac{l}{r}$ to 40 corresponding to a stress of 14 300 lb. per sq. in., whereas, the slightly lower limit of 14 000 lb. per sq. in. is directly fixed in the railway specifications.

Again, in the same Article, unit stresses for cast iron are given, which material has been carefully omitted from the railway specifications; and the allowable stresses for cast steel in shoes and pedestals are made only 80% of those previously given in the railway specifications. In Article 204,‖ also, limiting ratios of 120 and 140 for $\frac{l}{r}$ are used, compared with 100 and 120 in the railway specifications, and in Article 301¶, the minimum thickness of material is made $\frac{5}{16}$ in., instead of $\frac{3}{8}$ in., as in the railway specifications.

Under present conditions of heavy and constantly increasing highway loads, there seems to be little reason why highway bridges should not be designed and built to satisfy just as rigid requirements as railway bridges; and all the differences mentioned should be harmonized by the revision of one or the other of these two specifications.

Although the complete standardization of specifications of this kind is of questionable advantage, it seems highly desirable that the standardizing process should proceed at least far enough to establish a logical ground plan and a terminology whereby the writers and users of such specifications shall speak a common language.

Among the specifications for highway bridges in use ten years ago, although written by engineers of the highest reputation and widest experience, it was impossible to find two which agreed closely on such fundamental matters as live loads, unit stresses, allowance for impact, or even in regard to shop workmanship or grades and qualities of steel; and if, as often happened, a bridge engineer was asked by a city engineer to indicate the loadings or stresses in common use in highway bridge design, no direct or simple reply could be

* Cons. Engr., New York, N. Y.

† Received by the Secretary, January 8, 1924.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 471.

§ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1384.

‖ *Loc. cit.*, p. 1385.

¶ *Loc. cit.*, p. 1386.

given, but rather it was necessary to explain that both these matters were so involved with each other and with allowances for impact that the specifications then in general use could only be compared on a basis involving all these matters in combination.

For instance, in the specifications for steel highway bridges written by the late Theodore Cooper, M. Am. Soc. C. E., in 1896, and revised in 1909, which, perhaps, have been more generally used and are better known than any others, unit stresses were based on tensile stresses of 12 500 lb. per sq. in. for live loads and 25 000 lb. per sq. in. for dead loads, in medium steel. Although the word, "impact," was not used, making the dead load unit stress double that for live load was equivalent to a uniform addition of 100% to the live load stresses.

On the other hand, in the specifications written by the late C. C. Schneider, Past-President, Am. Soc. C. E., for the American Bridge Company, in 1901, and perhaps ranking next to Cooper's in use and reputation, the unit stresses were based on a tensile stress of 17 000 lb. per sq. in., for both live and dead loads, in medium steel; and an addition of 25% to all live load stresses was required "to compensate for the effect of impact and vibration".

Likewise, in the specifications written by George F. Swain, Past-President, Am. Soc. C. E., for the Massachusetts Railroad Commission, "Revised August, 1908", the unit stresses were based on a tensile stress of 16 000 lb. per sq. in., for both live and dead loads in "structural" steel; and additions, varying between 10% and 40%, to the live load stresses in various listed classes of members are provided to "allow for impact and vibration"; with the further provision that "members which receive stress only from lateral or longitudinal forces may be proportioned with unit stresses 20% greater than those above specified."

It is obvious that these three specifications could only be compared intelligently or fairly by applying each of them to specific cases and could hardly be described as written in the same technical language.

Due to the valuable work of the American Society for Testing Materials and of the American Railway Engineering Association and, latterly, of this Society, there now appears to be substantial agreement as to requirements relating to the chemical and physical properties of materials and as to what constitutes good shop workmanship and good details; and substantial progress has been made toward the treatment of the subjects of live loads, impact, and unit stresses, each on its merits, so that changes can be made under any one of these headings without necessarily involving changes in the others.

It certainly seems logical that the live loads should be selected to allow accurately for the known and anticipated combinations of external forces that the structure will have to resist; that the provision for impact should be made to represent truly the increment of stress due to the method of applying the live load, over and above the static effect of the same loads; and that the unit stresses should be based on the known physical properties of the materials used and on no other considerations whatever.

From this point of view, therefore, it is gratifying to note that the unit stresses proposed in these highway bridge specifications are practically the

same as those proposed for railway bridges, and are also in substantial agreement with those most generally used for other structures; and although the proved physical properties of steel may justify the adoption of higher unit stresses, it is to be hoped that if any changes are made they will be made applicable to all structures alike.

The outstanding features which differentiate highway from railway bridges are obviously the differences in the live loads carried and in the conditions under which these loads are applied; and these differences should be reflected accurately in the clauses relating to live load and impact. These two topics, therefore, will be discussed in the order named.

Live Loads.—The live load requirements in these specifications seem unnecessarily complicated, tending to emphasize the necessity of co-operation in the various efforts being made to write a specification for live loads on highway bridges which can be recognized as simple, easily applied, and representative, within reasonable limits, of modern motor traffic.

Of the many discussions on this subject that have appeared recently in the *Proceedings* of the Society, those by Lewis E. Moore, Assoc. M. Am. Soc. C. E.,* discussing the paper by Harold D. Hussey, Esq., on "Proposed Loading for Highway Bridges", and by L. H. Shoemaker, M. Am. Soc. C. E.,† discussing the "Tentative Specifications for Steel Highway Bridges", have impressed the writer as offering the best basis for a specification meeting the requirements stated, and it is to be hoped that some agreement may be reached substantially on the following lines:

For Trusses and Main Girders: On each traffic lane of the roadway, a uniform load per linear foot and, in addition, one excess axle load placed where it will produce the greatest effect, with variations in width of roadway treated, as recommended by Mr. Hussey,‡ but with a suitable reduction factor for roadways accommodating more than two traffic lanes.

On each sidewalk, 80 lb. per sq. ft. for spans less than 100 ft. long and 60 lb. per sq. ft. for spans more than 200 ft. long, with proportional loads for intermediate lengths.

For Transverse Floor-Beams: On each traffic lane a typical motor truck, but with a suitable reduction factor for roadways accommodating more than two traffic lanes.

For Roadway Stringers, Floor-Slabs and Planking: One typical motor truck with wheels placed to produce maximum effect.

In regard to all these loads, it is assumed, of course, that provision would be made for different classes of traffic, but that the loads for bridges carrying light traffic would be expressed as percentages of those specified for heavy traffic.

In view of the possibility that a motor truck may run wild and climb even an 8-in. curb, it would also seem desirable to design sidewalk brackets and sidewalk floors strong enough to support a maximum axle load, not, of course, at working stresses, but, at least, without collapse, and a concentrated side-

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1910.
† *Loc. cit.*, December, 1923, p. 2073.
‡ *Loc. cit.*, August, 1923, p. 1031.

walk load of, say, 25% of the maximum truck axle load might well be specified for this purpose.

Impact.—Comparatively little is known about the dynamic effect of motor vehicles on highway bridges, and until the research work which has been undertaken in this field has yielded more data, it would seem necessary to regard specifications covering this topic as purely tentative.

It seems obvious, however, that little similarity can exist between the dynamic effect of steam railroad trains drawn by locomotives with unbalanced drivers, followed by a series of uniformly spaced axle loads, and the corresponding effect of the wheel loads of motor vehicles with rubber tires running on a more or less smooth roadway surface, in constantly varying order and spacing.

If the roadway surfaces are smooth and unyielding and the tires in good condition, the impact should be practically nothing; but with holes and bumps in the roadway, or with defective tires, the truck wheels can deliver hammer blows easily equal to the static effect of the wheel or axle load, and thus justify a 100% addition for impact.

Such hammer blows, however, from different truck wheels are not likely to occur at the same time or to produce a cumulative effect, and there would seem to be an even chance that the effects from different blows might neutralize each other. For these reasons, it does not seem sensible to cling to the old type of impact formula involving "loaded length of span" (developed to suit railway bridge practice) however modified, and, in the writer's judgment, it would be more rational to treat impact as a suddenly applied concentrated load. This point of view, coupled with the live load recommendations previously made in this discussion, would permit impact allowances to be made exclusively as percentages of the proposed excess axle load specified for trusses and main girders, and of the motor truck loads specified for the floor system. Therefore, it would require no separate calculation.

In closing, the writer wishes to renew his recommendation made in a discussion* of the Tentative Specifications for Steel Railway Bridges, in favor of the inclusion of rules for the design of steel beams and girders encased in concrete.

That recommendation was not adopted in the final draft of the previous specifications, and the subject has not been mentioned in those now under discussion, but still it seems pertinent to point out that construction of this type, particularly in bridges carrying city streets, over steam railroads with limited head-room, is bound to be used extensively, and that, in such cases, substantial economy in the weight of the steel members can be effected without any sacrifice of strength, by the application of entirely sound principles of design. Undoubtedly, the best way to cover this would be as part of a Section devoted to the design of concrete floors and the statement of principles governing the rational and economical use of concrete and structural steel in combination, but until the time arrives when such a Section is added, the inclusion of an Article covering this detail is suggested, at least for discussion, to read as follows:

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 579.

"Where steel beams or girders are encased in concrete and the bond between the two materials is made effective, the combination shall be designed as a composite beam, in which all shearing and tensile stresses and the portion of the compressive stresses due to loads acting before the concrete has set, are carried entirely by the steel; but in which compressive stresses due to loads applied after the concrete has set, are carried partly by the steel and partly by the concrete, in proportion to the ratio of the moduli of elasticity of the two materials; the stresses in both materials to be determined by their moments of inertia, the total stress in the steel not to exceed the unit stresses herein previously specified, the compressive stress in the concrete not to exceed 500 lb. per sq. in., and the modulus of elasticity of the steel to be taken as fifteen times that of the concrete."

R. W. MACINTYRE,* M. AM. SOC. C. E. (by letter).†—The writer fully agrees with Mr. Carpenter's contention‡ that it is a mistake to establish a minimum width of 12 ft. for any type of highway bridge, as the stalling of a motor truck would block single-line traffic unless sufficient clearance was provided for passing any such possible obstruction.

The most important traffic bridge in the Province of British Columbia carries two lines of traffic across the Fraser River at New Westminster; it has a highway width of 16 ft., reduced to 14 ft. 8 in. at the roadway surface, due to an 8-in. wheel guard on each side. The destruction of posts and railing by overhanging fenders of heavy motor trucks is an almost daily occurrence which presents a difficult problem, involving danger to pedestrians using the sidewalk immediately contiguous to the highway railing. This raises the question of the necessity of a minimum width of 18 ft. on important arteries of traffic, as compared with 16 ft.

Standard specifications for bridge flooring and pavement should undoubtedly be established. The importance of these features has apparently failed to receive its proper recognition, if one may judge by the unsatisfactory condition and heavy maintenance expenditure under this head on many highway bridges, especially in rural districts. On bridges too light for any type of paving, the writer has had successful results by using 2 by 4-in. or 2 by 6-in. edge grain cross-strips toenailed to joists and secured with 7-in. wire nails horizontally, surfaced with a coat of tar paint. On the Fraser River Bridge, referred to, which is structurally too light for a solid pavement, the original wood blocks were recently taken up and renewed with a laminated type of floor consisting of 2 by 4-in. edge grain cross-strips, supported on timber stringers, and covered with an asphaltic concrete wearing surface. This has given good results.

JASPER O. DRAFFIN,§ ASSOC. M. AM. SOC. C. E. (by letter).‖—If the proposed specifications¶ are to meet the greatest need, they should be elastic enough to serve the smaller localities as well as the larger ones, not only in regard to the loadings specified, but also in regard to design and proportioning of members.

* Acting Chf. Engr. of Rys., Dept. of Rys., Victoria, B. C., Canada.
† Received by the Secretary, January 14, 1924.
‡ *Proceedings*, Am. Soc. C. E., January, 1924, p. 106.
§ Asst. Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.
‖ Received by the Secretary, January 14, 1924.
¶ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1379.

It would be unwise to insist that the rigidity of design should be the same for bridges carrying light traffic as for heavy traffic. If the different conditions of use are not considered, the smaller country localities would be tempted to build cheap and commercially designed structures. Therefore, the proposed specifications should include all ordinary highway bridges; any general revision should aim at increased flexibility. In this connection, the suggestion of Mr. Steinman,* to adopt in modified form the plan presented by Mr. H. D. Hussey is open to objection.† This form would add a uniform load preceding as well as following the truck concentration. One of the advantages claimed is that "it would afford a better representation of actual conditions on bridge roadways, where any local concentration of heavy trucks is generally preceded as well as followed by vehicles of lighter weight." This condition will exist on city and other bridges where traffic is dense, but probably not on most secondary roads, and on many so-called primary roads. Although practice should be standardized to a reasonable degree, the engineer should not be prevented by too rigid load specifications from using his judgment in analyzing and handling local conditions.

Comments on different Articles in the specifications are as follows:

Article 10.—Width of Roadway.—It seems questionable whether a clear width of less than 16 ft. should be specified, but if it is permitted, it should be restricted to Class *C* bridges.

Article 12.—Dimensions for Calculations.—For riveted construction, the length of floor-beams should be taken as the distance from inside to inside of vertical posts. In such construction, the connection creates a partly fixed-end condition which seems to be sufficient to compensate for any increase in span due to the bending of the vertical. With hangers, the length should be taken center to center of trusses.

Article 105.—Concentrated Loads.—The moments produced by the typical truck proposed are smaller than those of many trucks in use; in his own designs, the writer is accustomed to consider 0.85 W on the rear wheels and the wheel base as 13 ft.

Article 106.—Highway Live Loads for Girders and Trusses.—"The trucks shall be placed so as to produce the most severe stresses." A diagram should be included to show the position of the trucks on the roadway, with reference to the center line of the bridge, otherwise one designer will consider two trucks as close together as clearances will permit, while another designer will place each truck in the center of its line of traffic. It will be unusual on a bridge designed for two lines of traffic, to have two trucks side by side moving in the same direction; the common case will be two trucks moving in opposite directions. This would seem to be the logical condition to specify.

Article 110.—Bending Moment in Stringers.—This Article is one in which the writer is much interested. Specifications in the past have shown a tendency to designate permissible stresses with great refinement, while the loads which cause the stresses, are not known with anything like the same degree of

* *Proceedings*, Am. Soc. C. E., January, 1924, p. 103.

† "Proposed Loadings for Highway Bridges", *Proceedings*, Am. Soc. C. E., August, 1923, p. 1031.

accuracy. This is due, of course, to the fact that the condition of heavy concentrated loads on highway bridges is a recent development. A study of the specifications of a number of highway departments shows a great variation in the assumed distribution of concentrated loads, in some instances as much as 100 per cent. It would seem, therefore, that this is one place in the proposed specifications where much discussion would be profitable, especially so because a large percentage of the weight of a bridge is in the floor system.

The proposed method of distributing the loads does not take into account the length nor the moment of inertia of the stringers. A load supported by a series of long, shallow beams will be distributed to the different beams more widely than when supported by a series of short, deep beams. In the case of a plank floor, the theoretical distribution may be obtained approximately by the method of multiple elastic supports.* Table 14 gives values computed by this method. The value tabulated is the load carried by a single stringer, due to two wheels, 6 ft. apart, and is given in terms of percentage of one wheel load; it corresponds to the value obtained by the relation proposed in Article 110. These computations involve certain approximations, especially in the selection of the different combinations which are used, and, therefore, no claim is made that they are exact; furthermore, as only one plank of the floor is considered, the effect of the remainder of the floor in distributing the load is not included. Although any individual value may not be exact for actual floor systems, yet Table 14 shows that the effect of the spacing of the stringers is only one of at least three factors in the distribution of concentrated loads.

It is impossible to express the values in Table 14 by any simple equation. The following gives a close approximation:

$$\text{Percentage} = \frac{150\ \sqrt{(d-3)\times(s-0.7\,d)}}{L+12} - \frac{0.2\,d}{s}\,[(7-d)\times(24-s)]$$

in which, d = the depth of the I-beam, in inches; s = the spacing of the stringers, in inches; and L = the span of the stringers, in feet. In practice, the second term in the equation will prove to be negligible, thus leaving a simple formula. The better method, however, is to plot the computed values as a series of curves and use these for designing.

Table 15 gives values obtained by considering two trucks side by side and 10 ft. apart, center to center, in the middle of the panel. As might be expected, the two trucks give higher values in many cases, than the single truck.

A comparison of some values obtained from Table 14 with some of those obtained by experiment at the Iowa Experiment Station† shows a fairly close agreement. This comparison is made in Table 16, in which the computed values were obtained from Table 14 by interpolation. Other test data in the writer's possession agree as closely with the values computed by this method, as those in Table 16.

It is not proposed that the values given be used for design purposes, but it is suggested that, in the light of these computations and their generally close

* "Modern Framed Structures; Part II. Structures and Secondary Stresses", Johnson, Bryan and Turneaure. 9th Edition.

† "Load Concentrations on Steel Floor Joists of Wood Highway Bridges", by T. R. Agg and C. S. Nichols, *Bulletin 53*, Iowa Engineering Experiment Station, Ames, Iowa.

agreement with experiment, an effort be made to devise some method of load distribution, which will more nearly approach actual conditions than that given in these specifications. A relatively small number of properly selected experiments might easily be made, which would check the equations or values finally adopted.

TABLE 14.—VALUES FOR THE MAXIMUM LOAD TRANSMITTED TO ONE STRINGER OF A SERIES BY TWO TRUCK WHEELS 6 FT. APART. FLOOR SUPPORTING LOADS ASSUMED AS ONE 3 BY 12-IN. PLANK.

(Values Are Expressed as a Percentage of One Wheel Load.)

Span of stringer, in feet.	Spacing of Stringers, in Inches.				
	12	18	24	30	36
	6-In., 12.25-Lb. I-Beam Stringers.				
12	30.2	35.6	46.5	52.0	64.1
14	27.6	31.1	42.0	46.0	58.8
16	25.1	28.1	39.3	41.7	55.3
18	23.1	25.7	37.2	38.0	52.3
	8-In., 18-Lb. I-Beam Stringers.				
12	37.0	48.2	58.0	67.5	76.2
14	33.6	41.9	51.7	59.2	70.1
16	31.0	36.8	46.7	53.5	65.1
18	28.7	33.0	43.5	48.0	60.7
	10-In., 25-Lb. I-Beam Stringers.				
12	44.4	58.0	69.0	78.0	86.0
14	40.0	53.0	62.0	72.3	79.6
16	36.2	47.3	55.8	65.7	75.1
18	34.0	42.7	52.7	60.4	72.3
	12-In., 31.5-Lb. I-Beam Stringers.				
12	51.0	66.1	77.4	84.5	89.8
14	45.3	60.0	72.5	79.0	85.8
16	41.0	54.7	64.6	74.0	82.4
18	38.0	50.2	60.5	70.1	77.3

The distribution of loads through concrete slabs to stringers is difficult to determine theoretically, but a study of tests already made induces the writer to believe that the values given by the formula, $\frac{S}{6}$, are too high for values of S greater than 4 ft. Where there are five stringers 6 ft. apart and two concentrated loads are placed directly above the interior stringers, it does not seem probable that the adjacent stringers will fail to support any load.

Article 201.—Unit Stresses.—The formula proposed for columns seems to be in a direction opposed to the general trend of the times. It allows higher

TABLE 15.—VALUES FOR MAXIMUM LOAD TRANSMITTED TO ONE STRINGER BY TWO PAIRS OF TRUCK WHEELS ON A LINE TRANSVERSE TO THE STRINGERS. TRUCKS 10 FT. APART, CENTER TO CENTER, AND TRUCK WHEELS 6 FT. APART, CENTER TO CENTER.

(Floor Supporting Loads, Assumed as One 3 by 12-In. Plank. Values are Expressed as a Percentage of One Wheel Load.)

Span of stringer, in feet.	Spacing of Stringers, in Inches.				
	12	18	24	30	36
6-In., 12.25-lb. I-beam Stringers.					
12	30.2	42.6	52.5	57.0	77.3
14	27.6	40.5	49.9	53.7	72.5
16	26.8	38.6	47.7	51.7	69.5
18	26.8	37.0	46.2	49.8	68.9
8-In., 18-lb. I-beam Stringers.					
12	37.0	48.5	59.0	67.6	83.6
14	33.6	45.6	55.6	61.0	80.7
16	31.0	43.1	52.4	58.0	78.0
18	28.7	41.5	50.6	54.5	74.7
10-In., 25-lb. I-beam Stringers.					
12	44.4	58.0	69.0	78.0	87.4
14	40.0	53.0	62.0	72.3	84.9
16	36.2	47.3	57.4	66.2	83.6
18	34.0	46.0	56.2	62.5	81.1
12-In., 31.5 lb. I-beam Stringers.					
12	51.0	66.1	77.4	84.5	89.4
14	45.3	60.0	72.5	79.0	88.5
16	41.0	54.7	64.8	74.0	87.6
18	38.0	50.2	61.5	70.1	83.7

TABLE 16.—COMPARISON BETWEEN COMPUTED VALUES OBTAINED FROM TABLE 14 AND EXPERIMENTAL VALUES.

(Span of Stringers, 16 Ft.)

Size.	Spacing of stringers, in inches.	Distribution Factor:	
		Table 14.	Experiment.
6-in., 12.25-lb. I-beam.....	12	25	25
	19	30	39
9-in., 21-lb. I-beam.....	24	51	50-52
	24	..	56*
	27½	56	52-55
	27½	..	57*

*These values were obtained with the ends of the planks not fastened to the outside stringers.

stresses for low values of $\frac{l}{r}$ and lower stresses for high values of $\frac{l}{r}$ than the formula proposed by the American Railway Engineering Association, or that of the Special Committee on Steel Columns and Struts.*

Article 301.—Thickness of Metal.—The minimum thickness of structural steel $\frac{5}{16}$ in. should be qualified so as to permit the use of standard channels which have a web thickness of less amount.

Article 328.—Plate Girders.—There should be a statement as to the area to be used in computing the section modulus, whether gross area or net area.

Article 413.—Web Stiffeners.—This Article should require a bearing of the web stiffener against the flange which supports the concentrated load.

HENRY W. TROELSCH,† M. AM. SOC. C. E. (by letter).‡—There are a number of requirements in these specifications,§ that, while desirable in themselves, seem to be unnecessarily severe, and that will add to the cost without any corresponding benefit. The development of highways is always limited by the funds available, and although it is not desirable on that account to use bridges of cheap or unenduring construction, specifications should permit every reasonable economy, so as to encourage development.

For example, Article 330 requires the bottom cover-plates of plate girders to extend the full length of the girder. This is contrary to the almost universal practice. The same Article requires other cover-plates to be of such length as to allow at each end, beyond the point theoretically required, enough rivets to enable the plate to take its proportional part of the total flange stress. The Railway Bridge Specifications‖ submitted by the Committee provide (Article 325) for only two rows of rivets at each end beyond the theoretical point, and girders designed on that basis have been satisfactory. There is always a tendency for the end rivets of a cover-plate to be over-stressed; a lengthening of the cover-plate results in a slight decrease in such over-stress, but does not remove it unless the cover-plate is made full length. The writer believes that if the plate is made 1 ft. longer at each end than the theoretic length, and close rivet spacing is used at the ends until the strength is fully developed, all necessary requirements will be met.

Article 312 requires compression members to be fully spliced, without allowance for milled ends, whereas the Committee's Railway Bridge Specifications require only a 25% splice in built chord members. The requirement of full splices is not only unnecessary, but results in cumbersome details and will sometimes increase the lateral dimensions of members to such an extent as to affect adversely the economy of the design. In this connection, attention may also be called to Article 417, which implies that, in some cases, milled ends are to be used to transmit compression.

The provisions concerning counters and members subject to reversal of stress are evidently based on the fact that, in the past, the life of bridges has

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

† Asst. Engr., Am. Bridge Co., New York, N. Y.

‡ Received by the Secretary, February 4, 1924.

§ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1377.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 471.

been limited by the inadequacy of these members. Previous practice in this respect has been to use a uniform load with, perhaps, a uniform impact allowance for all members; to make no increase in the sections of members because of reversal; and to make no provision for future increase in the live load. The specifications differ from such practice in four ways:

1.—The live load increases in intensity as the loaded length increases (Article 106).

2.—The impact coefficient increases as the loaded length increases (Article 113).

3.—Members subject to reversal of stress have the stress of each character increased by 50% of the smaller stress (Article 120).

4.—For reversals or counter-stresses, an increase of 100% in the live load is required with only a 50% increase in the unit stresses (Article 114).

The first two of these provisions correspond in a general way to the actual effect of the live load and impact, and should be retained. The provision for increasing the size of members subject to reversal of stress is a relic of the days when bridges were designed on the basis of the fatigue theory. The occurrence of maximum loading on a highway bridge applied so as to produce maximum reversal of stress in the web members is infrequent and requires no special increase of sectional area, such as results from the application of Article 120. Connections of members subject to reversal, however, are liable to become loose, especially where pins or bolts are used, and the rule given might well be retained for the design of connections.

The provision of Article 114 seems designed to take care of future increase of live load. It is not apparent, however, why the loading should be increased 100% and the unit stresses only 50%, and why the increased loading should be used only for counters or reversals of stress. If it is desired to obtain a balanced design under an anticipated future increase of live load, or to provide for the occasional passage of unusually heavy loads at an increased unit stress, whatever loads and unit stresses are adopted should be applied to all members of the structure, including the floor, and not merely to counters or members subject to reversal. As it now stands, Article 114 would increase the members to which it applies to an extent not warranted by the size of the other members.

In Article 113, the term, "floor-beam hanger", might be interpreted to mean either a hip vertical, or a member suspending a floor-beam from a bottom chord pin. In either case, there appears to be no reason why such a member should be designed with a larger impact allowance than the floor-beam itself. It is possible that this requirement is intended to provide for bending stresses in the floor-beam hanger; if so, the proper procedure would be to estimate the bending stresses and to design the member accordingly.

Article 118 provides for a longitudinal force equal to 10% of the weight of an electric railway train. The writer would like to know whether this percentage has been derived from steam railway practice or otherwise. On steam railways, the brakes on freight cars are usually adjusted so as not to lock the wheels when the cars are empty. This results in a low friction coefficient when

applied to the loaded train. On interurban electric cars, however, the weight of the empty cars is a large percentage of the total weight, and the brakes are adjusted to utilize about 90% of the car weight. As a result, the longitudinal force from braking may be as much as 20% of the train load.

In Article 201, the allowable stresses are expressed in "kips" per square inch. The term, "kip", is perhaps fairly well known in the United States, but these specifications will be circulated and studied in foreign countries, where American colloquialisms are not understood. A man in Argentina or Australia or Japan, if he tried to look up the word "kip" in any English dictionary would find ten or twelve definitions, but not "one thousand pounds". If it is imperative to use such a word, it would seem advisable to add a note giving its meaning. In Article 203, it may be noted, the specifications revert to the more usual terms of "pounds per square inch".

Article 201 provides that for cast steel shoes only 80% of the allowable unit stresses for structural steel shall be used. This is more severe than the Committee's Railway Bridge Specifications, which allow as high stresses for cast steel as for structural steel. In view of the satisfactory behavior of cast steel shoes, these higher permissible unit stresses should be amply safe for highway bridges.

In Article 203, the phrase, "When the unsupported length of the flange exceeds twelve times the flange width", might well be omitted, noting that for short unsupported lengths the compressive stress is limited by the provision of Article 329, namely, that the gross section of the compression flange shall not be less than that of the tension flange.

Article 306, on net sections, has been submitted to a number of engineers and bridge draftsmen, who find difficulty in agreeing as to its interpretation. The writer would suggest that the second sentence, beginning, "The weakening effect", be changed to read as follows:

"The net section on a zigzag line passing through two or more rivet holes shall be determined by deducting from the transverse section one hole for the first rivet and a strip of width, w, for each succeeding rivet, as given by the formula;

$$w = h - \frac{s^2}{4\,g},$$

in which,

w = width of strip to be deducted;
h = diameter of hole;
s = stagger, or longitudinal spacing between the rivet in question and the next preceding rivet on the zigzag line;
g = transverse spacing, or the distance between the gauge lines of the rivet in question and the gauge line of the next preceding rivet on the zigzag line."

In Article 308, the use of the word, "plates", raises the question whether the thickness of angles or other shapes is to be included in determining the maximum length of rivet. Why not say "the total thickness of metal", or use the term, "grip"?

Article 322 calls for bottom lateral bracing in all bridges except I-beam spans. It seems unnecessary to provide bottom bracing in short deck plate

girder spans which are provided with intermediate cross-frames. Such bracing is not required by the American Railway Engineering Association Specifications for railway spans of less than 50 ft. Moreover, these specifications take no account of the bracing effect of a floor of solid concrete or of buckle-plates. When such a floor is properly arranged to resist lateral forces, there is no need of additional bracing in the plane of the deck.

In Article 328, the phrase, "provided the web is properly spliced to transmit its bending moment", should be omitted, since such splicing is called for in Article 332.

Article 341, on railings, should require the railing posts and their supports, as well as the top rail, to be designed to resist a transverse horizontal force. Where the railing is attached directly to the steel work, there should also be some provision for adjusting the alignment in the field, so that the top rail will be straight in the finished structure.

Article 420 is entitled "Pins and Rollers", but the Article itself deals entirely with pins, and makes no mention of rollers.

PERIODIC FLUCTUATIONS OF RAINFALL IN HAWAII

Discussion*

By L. Standish Hall, Assoc. M. Am. Soc. C. E.

L. Standish Hall,† Assoc. M. Am. Soc. C. E. (by letter).‡—The author's attempt to show a periodic cycle in rainfall by means of Hawaiian records is far from convincing. He states that,

"This cycle [3.7-year cycle] is so prominent in Hawaiian records that its accidental discovery was the immediate cause of the study preceding this paper, and the preliminary compilation of records and comparison with Bigelow's curves, brought the adoption of 3.7 years (or 44.4 months) as its average period."

As the entire study is based largely on the adoption of this cycle, it would have been interesting if the reasons for selecting this particular period had been elucidated more fully.

The apparent sequence of this cycle may well be within the probable variation of the rainfall. Statistics which are subject to variation, are naturally subject to cycles or periods above and below the mean, and if a short period is selected, the data may appear to follow a definite cycle. However, with a longer period, this regular cycle may be lost entirely. The writer has noted this in experimenting with artificial records formed by drawing from a box numbers found in a frequency series. It will be noticed in Fig. 6§ that the cycle derived from the 36-year record is much less pronounced than that derived from the 16-year record. A hasty comparison of Tables 2‖ and 5¶ will show that there is usually a wide range in the rainfall of months having the same cycle number; for example, Month 1 in January, 1900, has a minimum rainfall of 2.46 in. and a maximum of 13.07 in. in February, 1911; Month 27 has a minimum in August, 1909, of 2.45 in. and a maximum in March, 1902, of 32.24 in.

The author's criterion for the statistical proof of the existence of a cycle, the results of which are given in Table 9**, is inconclusive; in nearly all cases, the variation from unity is within the probable error.

Referring to the forecast of rainfall given in Table 11††, the improvement made in the estimate by using the cycle is only slightly more than that obtained by using monthly averages. In fact, the improvement is so small that it

* Discussion on the paper by Joel B. Cox, Assoc. M. Am. Soc. C. E., continued from January, 1924, *Proceedings*.

† Asst. Engr., H. L. Haehl, San Francisco, Calif.

‡ Received by the Secretary, January 17, 1924.

§ *Proceedings*, Am. Soc. C. E., October, 1923, p. 1707.

‖ *Loc. cit.*, p. 1701.

¶ *Loc. cit.*, p. 1708.

** *Loc. cit.*, p. 1714.

†† *Loc. cit.*, p. 1718.

would hardly seem to warrant the large amount of labor involved in making the estimate by this method. The reason for this small improvement is easily explained. In Fig. 9,* the frequency curve for the month of August, 1923, for the East Maui water-shed is shown. This curve has been lowered below the normal curve for August to allow for the cyclic influence. It is evident that the possible range in variation of the rainfall for this corrected curve is nearly as great as for the normal curve; in other words, the residual influences affecting the variation in the rainfall for this month, which cannot be accounted for by the assumed cyclic influence, are much larger than the correction which has been applied. A frequency distribution will only show the range of variation of the function involved, and there is no means by which the value of the function to be expected at any specific future date can be determined. If a cyclic correction could be determined, which would eliminate a large percentage of the unknown causes of variation, estimates made by the author's method would be a great improvement over the method of averages. Unfortunately, however, this is not the case.

The author's method of filling out short-term records by comparison with the longer records at other stations, as illustrated in Fig. 5,† is a legitimate use of the frequency curves and one that has been utilized often by the writer.

Although it is a matter of minor consequence, it might be noted that the author's method of smoothing out the records, as used in Tables 7,‡ and 8§ is somewhat different from that used in Table 6‖. In Table 6, five consecutive months are involved in the computation of the smoothed average, whereas, in Tables 7 and 8, seven consecutive months are used.

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1717.
† *Loc. cit.*, p. 1706.
‡ *Loc. cit.*, p. 1711.
§ *Loc. cit.*, p. 1713.
‖ *Loc. cit.*, p. 1709.

COMPARATIVE TESTS ON EXPERIMENTAL DRAFT-TUBES

Discussion*

By Messrs. Lynn Perry, H. A. Hageman, C. P. Dunn, and Clarence M. Weston.

Lynn Perry,† M. Am. Soc. C. E. (by letter).‡—This paper is a valuable contribution to hydraulic engineering, representing one more step toward the solution of the draft-tube problem.

The late James B. Francis, Past-President, Am. Soc. C. E., found from actual tests that the draft-tube not only permitted the mechanical equipment to be placed above the water level in the tail-race without loss of head, but that the addition of a draft-tube actually increased the efficiency of existing plants. Therefore, draft-tubes have been considered an essential part of hydraulic turbine installations.

Several fairly complete and instructive series of tests have been conducted in efforts to determine the most efficient shape and proportions of draft-tubes, all of which have thrown some light on the subject and influenced subsequent designs to some extent. It appears, however, that none of the investigations has succeeded in obtaining all the measurements that designers need.

A series of laboratory tests was conducted under the direction of Mr. W. M. White, in Milwaukee, Wis., in 1913-16, the results of which were presented in a paper§ before the American Society of Mechanical Engineers, in May, 1921. As a result, the Niagara Falls Power Company made further investigations which ultimately led to the use of a certain type of so-called "Hydraucone Regainer", which is reported to have indicated a net efficiency of 93 per cent. It would be interesting to have the tests of the various types of draft-tubes made at Niagara Falls and the subsequent experiences presented for comparison in the discussion of this paper.

Some European engineers have developed a theory of draft-tube design based on the assumption that there is stream-line flow parallel to the axis of the draft-tube. This theory is then applied whether the tube is straight or of the elbow type. Although there is no certainty as to the conditions inside the draft-tube at various gate-openings, it is known that the actual conditions do not even approximate the foregoing assumption. That whirl does exist, there can be no doubt; and there is reliable authority indicating that the whirl is in the direction of the rotation at the periphery and, at the same

* This discussion (of the paper by C. M. Allen, M. Am. Soc. C. E., and I. A. Winter, Esq., published in *Proceedings* for November, 1923, and presented at the meeting of the Power Division on January 17, 1924), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Asst. Prof., Hydr. and San. Eng., Lafayette College, Easton, Pa.

‡ Received by the Secretary, December 4, 1923.

§ "The Hydraucone Regainer, Its Development and Applications in Hydro-Electric Plants," *Transactions*, Am. Soc. Mech. Engrs., Vol. 43 (1921), p. 255.

time, in the opposite direction near the axis and that the flow is not even approximately axial, when the speed of highest efficiency obtains in certain types of high-speed runners. Some designers have erred seriously in not taking into consideration this important factor.

The principal problem in draft-tube design is to decelerate gradually the water which has two types of velocity, axial and whirling. To accomplish this, the writer believes that the tube should have a straight axis, a prolongation of the axis of the runner and that the inside surface of the tube should be conical.

The water emerges from the runner a whirling mass, the axis of rotation of which is a prolongation of the axis of the runner, and any attempt to change the axis of this whirling mass will certainly result in turbulence, with a consequent loss of energy. Before the direction of flow is changed, the velocity of the water must be reduced in a tube of geometrical cross-section, the axis of which is a straight line, the prolongation of the axis of the runner. A long, straight, tapered tube will fill this requirement which is important to a designer in laying out the turbine setting.

The argument against a straight tube is, of course, the excessive excavation or the necessary length of tube in a horizontal plant; in short, the cost. For this reason, recent tube designers have been prone to turn the water at right angles to the shaft. The tubes tested by the authors, and by other recent European and American investigators, are particularly interesting in that they turn the water at right angles to the shaft abruptly and still develop an efficiency as high as can be expected from a well proportioned, straight tube.

At present, the public interest in hydro-electric development seems to be increasing, and the probability of a more active construction era appears imminent. The results of every series of tests on this subject are of vital interest to hydraulic engineers. Those who like the authors have been engaged in research work of great importance to the profession and have set forth the results of their investigations so completely and so impartially as in this paper, merit the sincere gratitude of the profession.

H. A. Hageman,* M. Am. Soc. C. E. (by letter).†—The tests made by the authors on twelve model draft-tubes represent an important addition to present knowledge of draft-tube design. Inasmuch as the tests related to a particular wheel installation, the paper must be regarded as an important pioneer work rather than as a basis for drawing general conclusions.

For this particular case, the tests brought out the following points:

1.—The decided superiority of the newer draft-tube designs over the "quarter-turn" type so widely in use a few years ago.

2.—The comparatively large changes in efficiency resulting from small modifications in the outline of a particular type of tube.

3.—The apparent advantage of a high central cone with types of tubes which permit its use.

The tests were apparently made with all the care and refinement necessary, and their accuracy is not questioned. It would be interesting, however, to have

*Hydr. Engr., Stone & Webster, Inc., Boston, Mass.

† Received by the Secretary, December 11, 1923.

an explanation of the method used in computing effective head on the water wheel. It is stated* that the effective head was measured directly by means of a gauge which indicated the difference between head-water and tail-water levels.

The usual practice in determining water-wheel efficiencies has been to deduct from the total head acting on the turbine, the head corresponding to the discharge velocity at exit from the draft-tube. It is not clear whether this method was followed.

The new test code for hydraulic power plants of the American Society of Mechanical Engineers apparently does not require the deduction of velocity head at exit from the draft-tube. If these tests were made in conformity with the new code, it should be so stated, as this code is not recognized at present by all water-wheel manufacturers.

In studying the effect of decreasing the discharge area of a draft-tube, as shown in Fig. 24,† the consideration mentioned is of some importance, inasmuch as the velocity head at exit, for the smaller discharge areas, becomes an appreciable proportion of the total net head.

These tests open an interesting field, and it is to be hoped that such work can be continued. It is suggested that valuable data could be secured by making a similar series of tests on the same tubes, using a runner of widely different specific speed.

C. P. DUNN,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—This paper is of rare excellence, because it completely and concisely tells the story of the tests. It is difficult to discuss and add to it unless data pertaining to similar tests at present unpublished, are made available.

The writer wishes to stress the second paragraph of the Synopsis, in which the authors point out the fact that this paper records the results of one specific problem, and, therefore, cannot be considered as embracing the entire range of draft-tube design. The science of hydraulic turbine design has advanced so rapidly in the past few years, both in respect to size and to efficiency of units, that it is not surprising if the study of this accessory—the draft-tube—should have lagged behind. It is to be hoped that those in a position to make experiments with draft-tubes will carry the study further in the near future. The writer appreciates the money value of apparently insignificant items connected with the efficiency of generating units. He has had opportunity to observe installations where marked increase in output can be obtained by making slight, inexpensive changes in old units, principally by the substitution of runners of modern design for the older ones.

The name, "draft-tube", handed down from the early days of turbine design, does not convey a complete idea of the true function of this accessory to the modern turbine. It is, of course, used in part as a chamber or tube where the pressure is below atmospheric, in order that the ordinary turbine may be set at a convenient elevation as regards its construction and operation. This func-

* *Proceedings*, Am. Soc. C. E., November, 1923, p. 1822.

† *Loc. cit.*, p. 1836.

‡ Designing Engr., Portland Ry., Light & Power Co., Portland, Ore.

§ Received by the Secretary, December 26, 1923.

tion of the draft-tube appears to be indefinite when one considers that many wheels, at high tail-water stages, are actually submerged, and that others, on account of the high velocities obtaining immediately below the runner, must of necessity be at such an elevation that there is little or no "draft" head. The draft-tube might, with greater accuracy, be called a "decelerating chamber".

As it appears to the writer, the real function of the tube is the efficient change of velocity head in the water as it leaves the runner to static head, capable of doing useful work.

It is probable that a valuable study could be made of draft-tubes by using curves showing rate of reduction of "velocity-head" energy in the draft-tube, based on the performance of actual or experimental tubes. Leaving the true "draft" or "suction" head out of the problem, it should be possible to determine the most economical length as well as the most economical shape. A study of the "rate of reduction of velocity head" curves would show the most economical shape of such curves, which, in turn, would determine the shape of the "flare" of the draft-tube.

The writer appreciates the fact that partial data pertaining to these questions are available in the authors' paper; it is impossible, however, to draw any such curves as those mentioned without the complete dimensions of the tubes.

Clarence M. Weston,* Assoc. M. Am. Soc. C. E. (by letter).†—In making a study of these tests with special reference to their bearing on the design of the simpler forms of the elbow type of draft-tube, several questions have arisen which the writer would like to discuss.

It is assumed that the net head of 1 ft., as used in plotting the final results, is the difference in the elevation of the head-water and tail-water. If to this a correction for the velocity head at the outlet of the draft-tube is applied in accordance with the definition of the net effective head on a unit, as given in the testing code approved by the Machinery Builders Society in 1917, it will be found that for the draft-tubes listed as Types L-5 and M, the net effective heads would be 0.990 and 0.998 ft., respectively. The maximum efficiencies for these two draft-tubes as taken from the test curves is 86.7% and 87.5%; dividing these figures by the ratio of the net effective heads to the net heads as used, gives 87.6% in each case. In other words, when using the net effective head on the unit, as defined in the code mentioned, the efficiencies of the two draft-tubes are the same.

The writer would like to inquire if there is anything in these tests to indicate that a draft-tube similar to Type L-5 will not give just as high an efficiency as one of Type M in cases where the runners can be set a little higher above the tail-water, thus providing space for a draft-tube similar but larger—one in which the velocity head at the outlet would be about the same as with that of Type M, and also giving the vertical distance required for the flare connecting the turbine ring to the larger tube.

Type M is a relatively wide tube, which factor would often determine the spacing of the units in a power-house, whereas Type L-5, even after increasing

* Designing Engr., New York, N. Y.

† Received by the Secretary, December 27, 1923.

the outlet area to equal that for Type M, would only require a spacing of about 0.8 as much.

In the case of Type L with which tests were made to determine the effect of flaring sides in the horizontal section of the draft-tube, it will be noted that in all cases except Type L-5 the flare is more than 4° and that the loss due to the flare is greater than the head regained by the reduced outlet velocity. Although the variation in the efficiency is small, yet it would tend to confirm the old assumption that 4° is about the proper limit of horizontal flare.

Referring to the Type E draft-tube on which tests were made with and without an extension on the discharge end, it is interesting to note that with one size of throat, after correcting for the difference in the velocity heads at the outlets, the efficiency is the same whether or not the extension is used. This is true for each of the two sizes of throat used and might be considered as a check on the fine accuracy of the experiments.

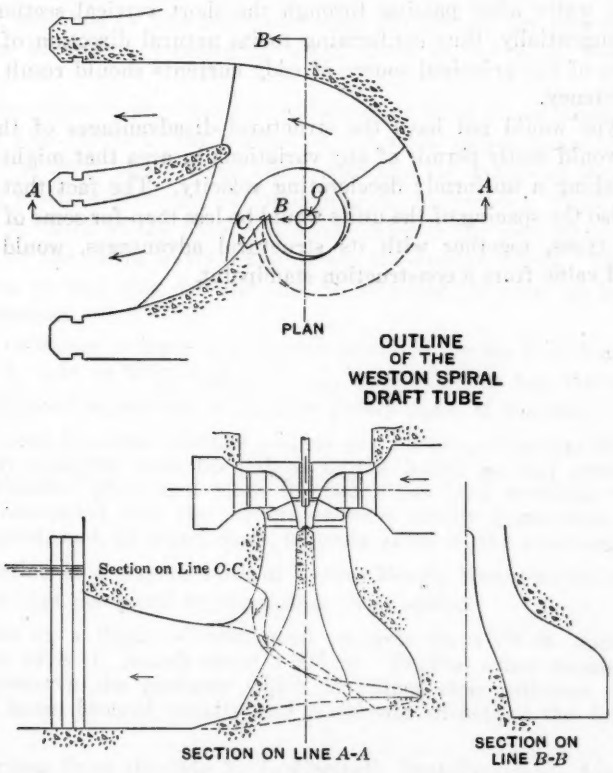

FIG. 20.

It is noticeable that all the draft-tubes included in these tests are symmetrical about a vertical plane passing through their center lines and do not conform to the action of the water as it passes through. As the water leaves the turbine runner, it has a whirling motion which is not entirely overcome before it leaves the outlet. It has been observed in carefully designed draft-

tubes of both the elbow and the spreading types that the water discharges considerably faster on one side than on the other, and, in some cases, actually flows back into the draft-tube on one side. This would indicate that even greater efficiency may be expected when a design is evolved which conforms more nearly to the natural flow of the water.

With this thought in mind and realizing the increasing importance in the development of the high-speed runners of regaining as much as possible of the velocity head contained in the whirling water, the writer believes that a draft-tube embodying the principle of the scroll case in reverse form would be one of the most logical and most promising types for future study and experiment. Although various studies have been made on draft-tubes of this type, the writer has been unable to obtain definite information regarding any that have been designed on a practical working basis for vertical units.

A spiral draft-tube similar to the one shown in outline on Fig. 30 would permit the water after passing through the short vertical section to be discharged tangentially, thus conforming to its natural direction of flow. This elimination of the principal source of eddy currents should result in a higher rate of efficiency.

This type would not have the structural disadvantages of the spreading type and would easily permit of any variations in area that might be required for establishing a uniformly decelerating velocity. The fact that the excavation and also the spacing of the units would be less than for some of the recently developed types, together with its structural advantages, would add to its economical value from a construction standpoint.

OCEAN BEACH ESPLANADE, SAN FRANCISCO, CALIFORNIA

Discussion*

By Lewis M. Haupt, M. Am. Soc. C. E.

Lewis M. Haupt,† M. Am. Soc. C. E. (by letter).‡—The author is to be congratulated for the thorough manner in which he has reported the methods used and results obtained in this particular case illustrating the ever-present problem of shore protection, concerning which there are so many divergent opinions.

In this solution there is provided a total barrier of 42.75 ft., of which 27.25 ft. is below and 15.5 ft. above the surface of the sand, under normal conditions. The crest is 22 ft. above mean high water and the radius of the curve is 7.5 ft., giving a slight overhang at the coping of the parapet, which is 3.5 ft. high and 2.0 ft. thick at the pavement level. This is the part which must resist the shock of the storm waves when driven by onshore winds. No man can foretell when or how this defense may be attacked; it must be left for the records to disclose.

To this end, reference is made to a digest§ compiled for the Pan-American Congress of 1915, held at Washington, D. C., from which a few statements are submitted. Regarding erosion of the New Jersey coast, it is stated:

"No general plan has been pursued and no general protection has resulted * * *, but great damages have been done to the buildings and properties behind the bulkheads. More and more the beach has been receding * * *, so that it was concluded that the structures were wholly inadequate, were improperly designed, and, in many cases, actually aided in the coast erosion."

Referring further to a type of work at Revere Beach, Mass., which is similar in design to that described by the author, it is stated:

"This consists of a flight of reinforced concrete steps 30 ft. wide and rising a distance of 8 ft., length about 1 500 ft. Various other means have been tried to preserve the parkway which is immediately adjacent to the shore, but they have decayed rapidly and could not withstand the force of the seas."

It further quotes from the late E. L. Corthell, Past-President, Am. Soc. C. E., as to the Holland dikes on the Island of Westkapelle, the details of which are cited in this discussion, and adds that:

* Discussion on the paper by M. M. O'Shaughnessy, M. Am. Soc. C. E., continued from February, 1924, *Proceedings*.

† Cons. Engr., Physical Hydrography, Cynwyd, Pa.

‡ Received by the Secretary, November 24, 1923.

§ By the late B. F. Cresson, Jr., M. Am. Soc. C. E., then Chief Engineer, Harbor Comm., State of New Jersey.

"The policy has been to use only one dike, keeping it as close to the sea as possible, resorting to a second line of dikes only when every other means have failed. In 1682, the dike was broken and the sea flowed in for five successive tides before it could be repaired. * * * Conditions recently have been more serious owing to the constant deepening of the sea bottom at the foot of the dikes. * * * In 1911, a stone weighing 6 tons was carried more than 30 ft. and thrown over a row of piles."

Fig. 10* gives a graphic view of the situation along the coast of Zeeland, concerning which it has been stated,† "Even these groins have not always proven successful as some sections of the coast, although protected, have continued to decrease." It is also officially reported that both high and low groins had little value and that a new system was introduced as the situation was "becoming very serious, and in 1898 the paving was introduced [Fig. 11], which proved more efficient," but was expensive. "The groins were 600 m. long, built of piles and stone, but they had to be renewed every 5 or 6 years so that reinforced concrete had to be substituted, but it cannot be used everywhere."

It is also stated that the inner parts of the groins were built of concrete and the outer of fascines and stone. "In 1915, the land end of some was washed out by the tidal currents followed by the undermining of the concrete layers and the breaking down of the whole construction."

It has been recommended that "bulkheads seaward of the high-water line should not be permitted, except for urgent reasons," yet the experience on many coasts indicates that even when placed on, or landward of, that location, they are impotent to prevent the ravages of the sea.

A striking instance is noted by Mr. W. T. Douglass, of Great Britain, where several efforts were made to resist the attack of the sea by building a concrete wall, at first 20 ft. deep, and, later, a second one, 30 ft. deep, both of which were destroyed without any apparent reduction in the rate of erosion (Fig. 13); on the other hand, where an offshore sea-wall was built at Ceará, Brazil, as a landing pier and harbor, connected with the shore by an open viaduct, it soon after completely closed the harbor and advanced the shore-line to its outer end.‡

The sea-wall at Galveston, Tex., designed by a board of three most competent engineers, was placed on the high-water line, and was founded on a grillage, above which it rose 16 ft., with the expectation that the seas would never surmount it; yet, they have swept across the island and destroyed a part of the connecting causeway (Fig. 12). A number of spur jetties were built of timber and rock, all of which have been destroyed, and the footing of heavy granite has to be replaced constantly to protect the toe (Fig. 14).

A sea-wall, at Longport, N. J., similar to the one at San Francisco, Calif., but lower, has been so badly breached (Fig. 15) as to have denuded the properties abutting against it.

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* From the description of Joseph J. De Kinder, C. E., of Holland, entitled "The Garden of Zeeland."

† *Engineering Record*, January 22, 1916.

‡ "Beach Erosion; Its Causes and Cure", by Henry Clay Ripley, M. Am. Soc. C. E. *Proceedings*, Am. Soc. C. E., January, 1924, Fig. 2, p. 17.

FIG. 10.—SHORE PROTECTION NEAR ZOUTELANDE, HOLLAND, DIKES WITH RIP-RAP AND PILES AND LONG JETTIES.

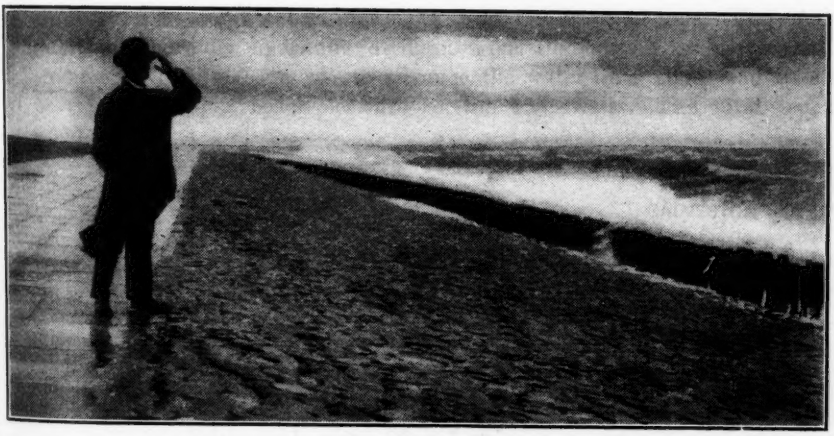


FIG. 11.—WEST KAPELLE DIKE, HOLLAND. BROAD, FLAT, PAVED SLOPE, PROTECTED BY A PILE BARRAGE AT THE TOE.

FIG. 12.—GALVESTON, TEX., SEA WALL, 1918. THIRTY-FIFTH STREET LOOKING WEST. RAMP CANNOT BE USED ON ACCOUNT OF EROSION.

Many other instances might be cited to illustrate the force of the storm waves beating on a resisting barrier located at or near the high-water line; the effect on the structures may be appreciated from a study of Fig. 16.*

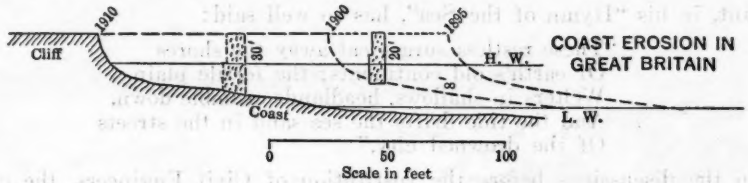


FIG. 13.

Another instructive example of the effort to reclaim and extend the beach front will be found at Coney Island, Brooklyn, N. Y., where a series of jetties were placed at intervals of 600 ft. They are about 300 ft. long, the inner portion being made of a double row of lapped piling and the outer ends, for about 200 ft., are built of stone rising to 2 ft. above high tide and are 10 ft. wide. The water at the outer end is 15 ft. deep and the width of base is 50 ft. The stones range in weight from 100 lb. to 10 tons. The sand was pumped ashore by hydraulic dredges, from the bottom of the sea, to the amount of 1 700 000 cu. yd., to advance the water-lines about 250 ft. and to provide a bathing beach seaward of the boardwalk. The total length is 9 500 ft.

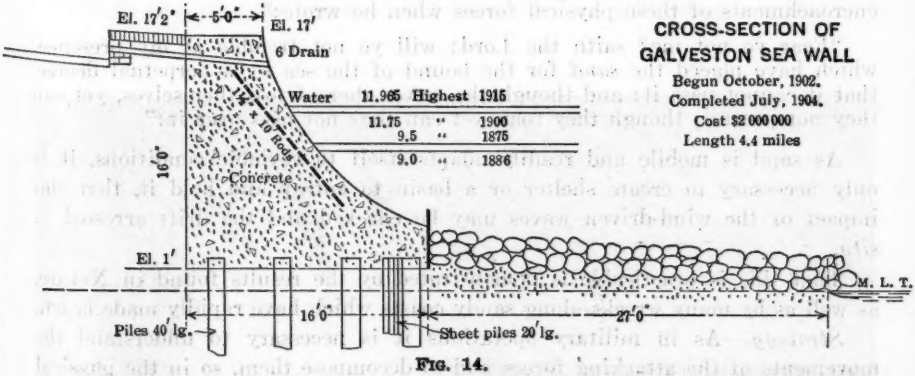


FIG. 14.

The storms of October, 1923, are said to have driven the sand shoreward nearly to the boardwalk and to have caused some local erosion, but as this beach was deposited artificially the equilibrium was not determined by the method of construction, but by the resultant of the physical forces, thus furnishing an interesting example for the solution of this ever present problem. (See Fig. 17.) The great popularity of this resort and its proximity to the dense population of New York and vicinity have fully justified the cost of this great improvement, completed in 1922.†

* See, "Coast Erosion and Protection", by E. R. Mathews, Lond., 1913.

† Additional information on the reclamation of the Coney Island beach may be found in the report of the Brooklyn Engineers' Club of 1914, published in response to a competition for prizes offered by Mr. Alfred T. White, which it would be redundant to repeat in this discussion, yet, it would seem that the question is still open for further consideration, as to the best and cheapest remedy.

Fundamentals.—The elements which enter into the changes which are taking place on the seaboard are the winds, waves, tides, and currents, operating in conjunction or in opposition, concerning which William Cullen Bryant, in his "Hymn of the Sea", has so well said:

> "These restless surges eat away the shores
> Of earth's old continents; the fertile plain
> Welters in shallows, headlands crumble down,
> And the tide drifts the sea-sand in the streets
> Of the drowned city."

In the discussions before the Institution of Civil Engineers, the expert opinions prevailed, that:

"The object of a sea-wall should not be to arrest the impact of the waves, but to form a barrier against which the shingle, sand, or other drift along the coast might be arrested and so become the real protection. A sea-wall is not a true remedy but only a palliative, by substituting a harder material for a softer."

As man is impotent to control the activity of the forces, it remains for him to select and place the materials best adapted to resist their injurious effects. For this purpose it is frequently assumed that rock, being heavier and more durable than other material, is the best, yet its weight is a serious objection and the cost is much greater than that of wood or sand.

The prophet Jeremiah clearly indicated the best barrier to prevent the encroachments of these physical forces when he wrote:*

"Fear ye not me? saith the Lord: will ye not tremble at my presence, which have placed the *sand* for the bound of the sea by a perpetual decree, that it cannot pass it: and though the waves thereof toss themselves, yet can they not prevail; though they roar, yet can they not pass over it?"

As sand is mobile and readily adapts itself to variable conditions, it is only necessary to create shelter or a basin to entrap and hold it, that the impact of the wind-driven waves may be checked and the drift arrested *in situ.*

That this is practicable is demonstrated by the results found in Nature, as well as by many wrecks along sandy coasts which have rapidly made beach.

Strategy.—As in military operations it is necessary to understand the movements of the attacking forces and to decompose them, so in the physical domain, the policy should be to locate the resisting barriers so as to prevent assault on the main line, that is, by outposts, to prevent a direct charge over the top, or by a flanking movement. Hence, it is unwise to locate the protective work directly on or within the high-water line, where it must receive the full force of the wind-driven waves, as is so strenuously asserted by some physicists. Instead, the maritime assault should be decomposed and retarded before it breaks over the inner lines, by using offshore barriers of such forms as to resolve the waves into their components and cause the breakers to cushion on the water instead of on the sandy sloping beaches, thus also creating a lee for the protection and deposition of the sediment with which they have become charged.

* Book of Jeremiah, Chapter V, 22.

FIG. 15.—SEA WALL AT LONGPORT, N. J., AFTER STORMS OF FEBRUARY 4-6, 1919.

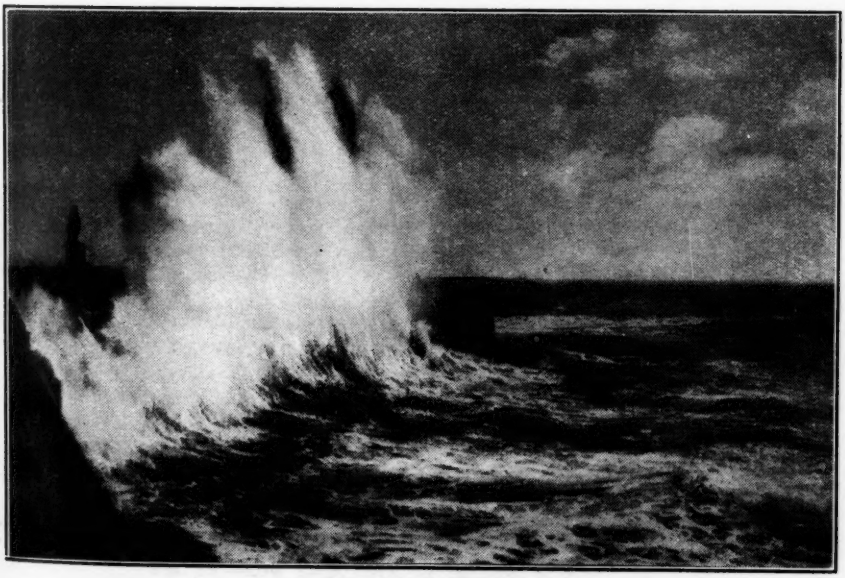


FIG. 16.—VLISSINGEN, HOLLAND, IN A SOUTH WESTER, SHOWING FORCE OF WAVES. NOTE JETTY EXTENDING OUT FROM SEA WALL.

FIG. 17.—CONEY ISLAND, BROOKLYN, N. Y. BEACH AND BOARDWALK, SHOWING JETTIES 600 FT. ON CENTERS, PROTECTING BEACH, AS ON FEBRUARY 23, 1923.

FIG. 18.—LOW SPUR, 1918, AT BELMAR, N. J.

The general direction of the resultant movements must also be observed carefully in order that the enceinte may be properly co-ordinated to arrest the progressive drift and impound it. This involves a study of the general direction of the coast contours with reference to the angle of tidal approach, and the slope of beach, also, local features, such as the proximity of inlets or capes and the character of the materials in place, whether rock, meadow sod, coarse gravel, shingle, or sand; but the most important factor in the littoral transportation of the drift is the prevailing angular direction of the breaking waves, which necessarily is a function of the winds.

Equipped with a knowledge of these elements, the plan of protective works may be prepared with reasonable promise of success, as will be seen from the evidence of works which have withstood the most violent storms for years, and yet in which no stone nor brush has been required, although, in some instances, they have been used by contractors to make assurance doubly sure.

The Testimony of Nature.—There is no better evidence of the resultant effects of these forces than that furnished by Nature; no verbal descriptions can portray them so well as graphic demonstrations giving the environments.

Fig. 18 is a low jetty (improperly called "groin") crossing the strand, which has arrested the northwardly drifting sand on its windward flank, denuding the leeward side and causing the surf to cascade over, thus washing down the sand to be driven again to the next jetty. By such a series of hurdles, the drift traverses the shores giving little or no protection from storm tides.

Figs. 19 and 20 indicate the futility of attempting to check storms by wooden pile bulkheads, which, in many instances, have been ruptured and dwellings undermined and destroyed. At the south end of this beach, at Rehoboth, Del., Fig. 21, a wrecked barge was stranded several years ago, which has created a large deposit of sand and held it to date, as has frequently happened elsewhere; it constitutes an offshore barrier to create a lee and check the waves.

The east coast of England has lost about three miles since the Roman invasion and, yet, after some years of investigation, it has been decided that the protection of its littoral is not a function of the Government, but of the riparian owner. Moreover, it is stated that:

"If one portion of a beach was protected by jetties the immediate result would be the denudation of the portion to leeward, yet it would be improper for a seaside resort to let the beach sand drift by, merely because by retaining it they might be robbing somebody else."

Thus, although the injurious effects of the normal, tight jetties are recognized, until recently, there was no alternative. As the cost of protection by sea-walls and jetties is prohibitive in most cases from the standpoint of the riparian owner, nothing is done and the revenues to the State and nation are lost.

Another illustration of the inefficiency of spurs in the nature of rocky headlands is seen in the cliff at Tynemouth, England, Fig. 22, where the detrusion has not prevented the destruction of the priory and other buildings on the coast, nor has any sand been deposited to arrest the waves.

At Coronado Beach, Calif., where the sea was cutting the alluvial bluff rapidly and encroaching on the driveway, a wooden bulkhead was erected in front of the bluff, but was destroyed. Similar structures elsewhere have met the same fate even when built of interlocking steel piles in front of a bulkhead, and no beach has been reclaimed.

Many other instances of the futility of attempting to protect riparian lands from the encroachments of the sea by use of barriers on or within the high-water line might be cited, but the economic question of cost is an important factor, as well as the efficiency and maintenance of the structures. The cost of $130.72 per lin. ft. given by the author is manifestly beyond the limit of most riparian owners and must needs be underwritten and financed by public administrators.

One of the most successful instances of private protection is found on the New Jersey coast where a series of five bulkheads was built to arrest the erosion of the bluff. These parallel barriers covered a frontage of 300 ft. and their interspaces, forming pockets, were filled with rip-rap constituting a succession of wave-breakers (Fig. 23). There is no bathing beach remaining, so that this construction does not seem to fulfill the required conditions for a summed resort by the seaside.

Another instance of the futility of bulkheads placed on the high-water line is found in the complete destruction of the buildings at Galilee, N. J., by the storm of 1918, as shown in Fig. 24, which tells the story fully.

In view of these and many other precedents, and the great cost for construction and maintenance, it is desirable to submit the evidence of the results obtained by structures which, without robbing the beaches, utilize the physical forces and construct their own sandy barriers at a minimum expense, and which have withstood the onslaught of the most violent storms.

Automatic Remedial Measures.—In accordance with the fundamental principles of strategy in resisting destruction, it is found that barriers placed across the strand or even beyond the low-water line have so demoralized these combined forces as to cause them to deposit the sand with which the breakers become charged, behind the barriers erected to retain it, in such manner as not to rob the beaches to leeward by using tight spurs, but merely to check the drift on both flanks and thus build up a barrage of sand which rapidly protects the coast and recovers the lost riparian property at a minimum cost.

The particular form, length, position, and grades of these works must be adapted to the local physical conditions and resultant movements of the littoral drift. At Barnegat City, N. J., for instance, the lighthouse was saved, in 1920, when the Borough built a hooked jetty only 150 ft. long, reaching into water 19 ft. deep, which soon became dry land, at a cost of $2 500; a rip-rap jetty, composed of brush and heavy stones and supplemented by concrete retaining walls, was impotent to check the storms of 1919, as shown in Fig. 25, but, subsequently, the curved jetty proved to be successful, as indicated in Figs. 26 and 27. In 1912, a few curved jetties were built at Ventnor, N. J., which recovered and maintained the bathing beach, and are now buried almost out of sight, with no cost for maintenance. There are many other

FIG. 19.—REHOBOTH BEACH, DEL., SHOWING WOODEN PILE BULKHEAD INEFFECTUAL.

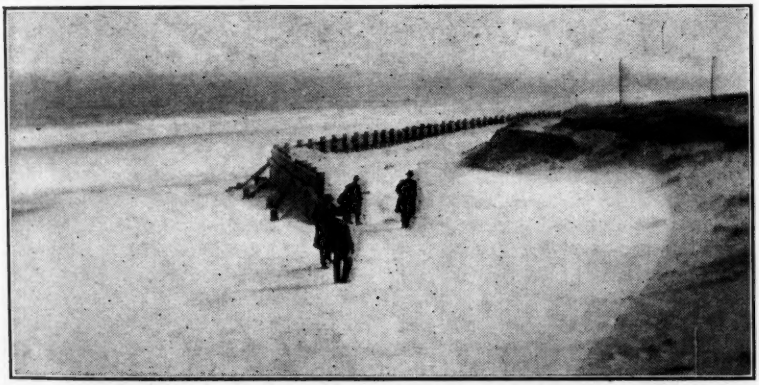


FIG. 20.—REHOBOTH BEACH, DEL., PILE BULKHEAD.

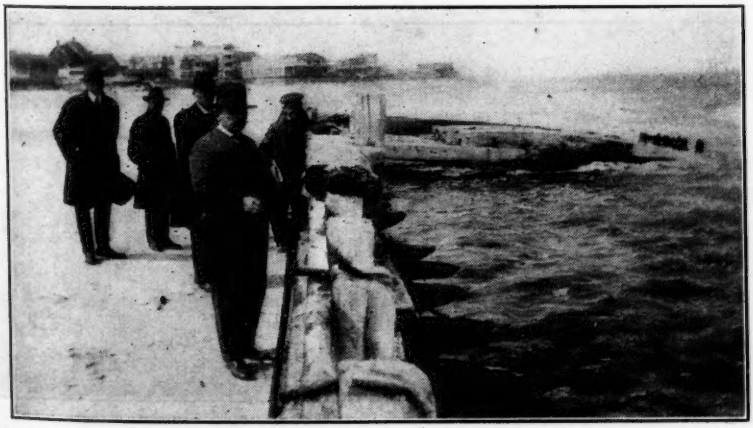


FIG. 21.—REHOBOTH BEACH, DEL. EFFECT OF WRECK OF BARGE IN MAKING BEACH. WRECK OCCURRED IN 1918; PHOTOGHAPH TAKEN MARCH, 1922.

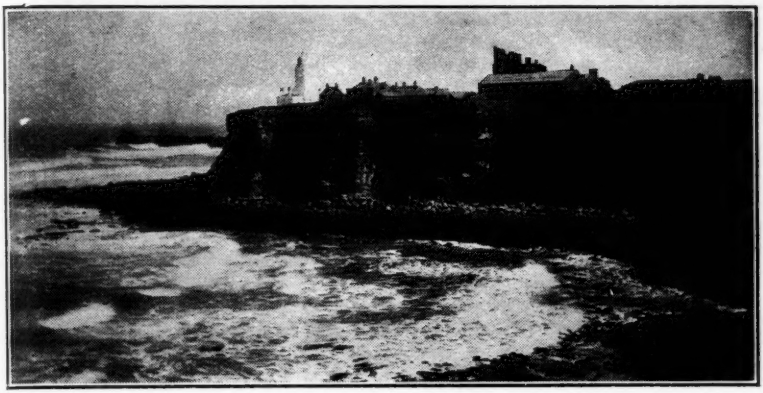


FIG. 22.—TYNEMOUTH, ENGLAND, SHOWING ENCROACHMENT OF SEA AGAINST ROCKY HEADLAND.

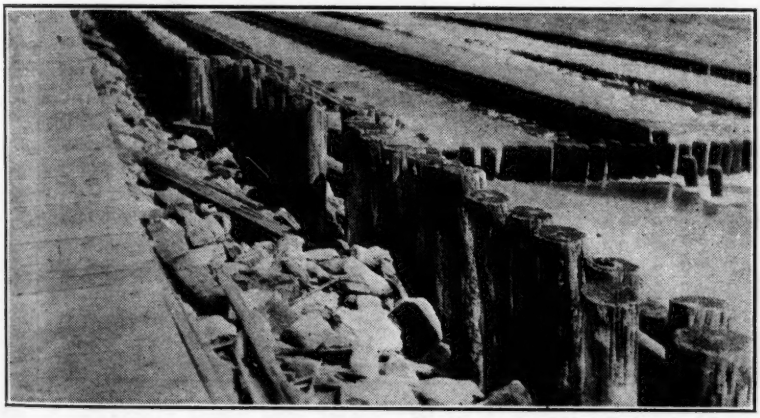


FIG. 23.—LOW MOOR, N. J., SERIES OF LONGITUDINAL BULKHEADS. BEACH USELESS.

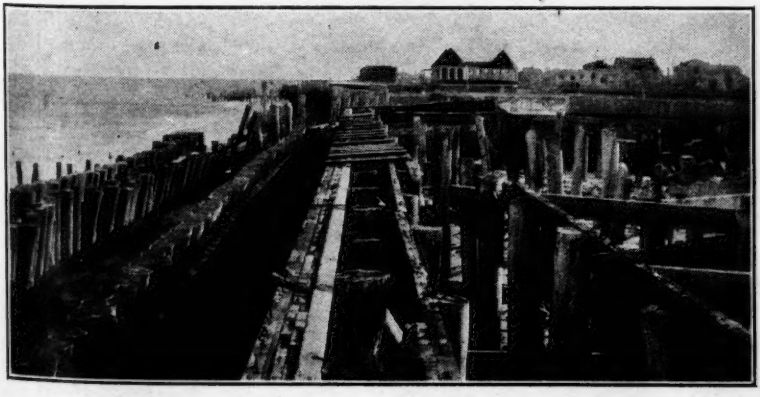
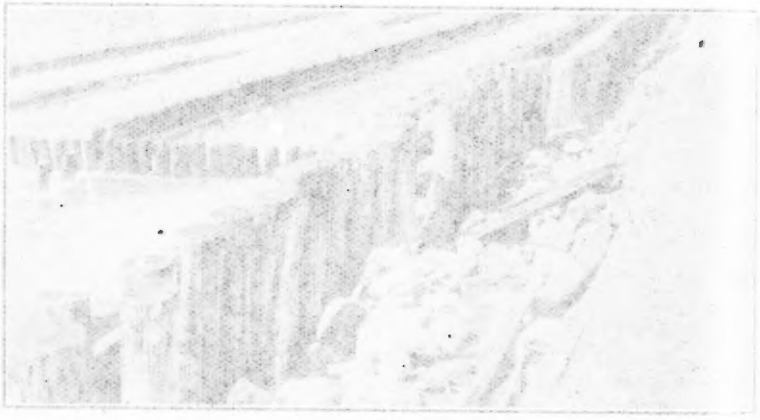


FIG. 24.—GALILEE, N. J. BULKHEADS AT HIGH-WATER LINE FAILED TO PROTECT.



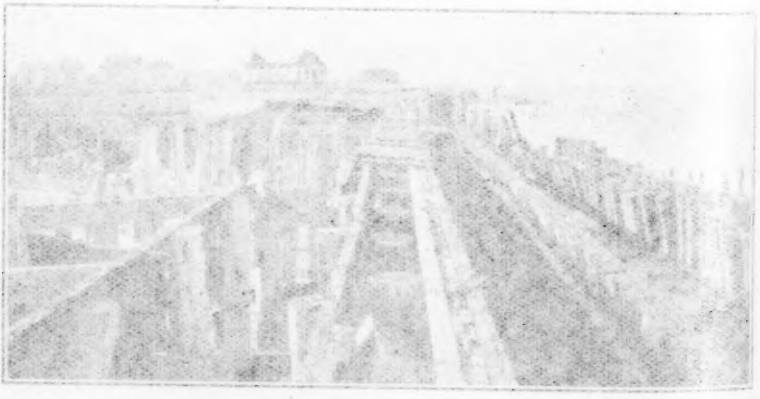


FIG. 25.—BARNEGAT LIGHT, NEW JERSEY. FAILURE OF ROCK AND BRUSH JETTY AND CONCRETE WALL.

FIG. 26.—BARNEGAT LIGHT, NEW JERSEY. BEACH RECLAIMED BY WEST JETTY IN 1920, SHOWING PRECARIOUS POSITION OF LIGHTHOUSE.

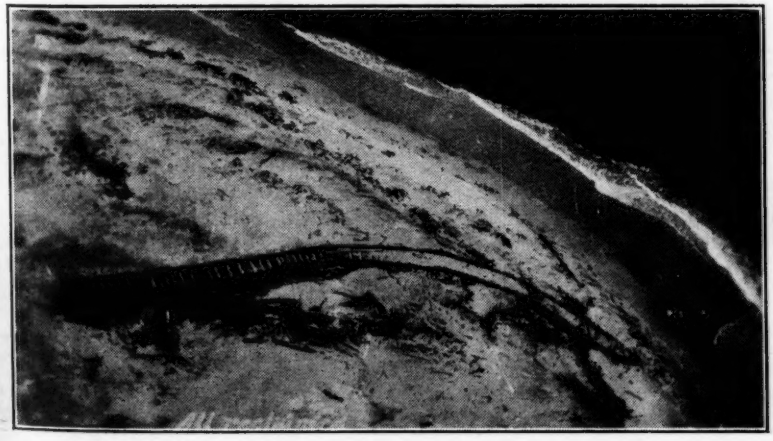


FIG. 27.—BARNEGAT LIGHT, NEW JERSEY. BIRD'S-EYE VIEW OF WEST JETTY FROM TOP OF LIGHTHOUSE. SHOWS EXTENT OF BEACH RECLAIMED IN A FEW MONTHS.

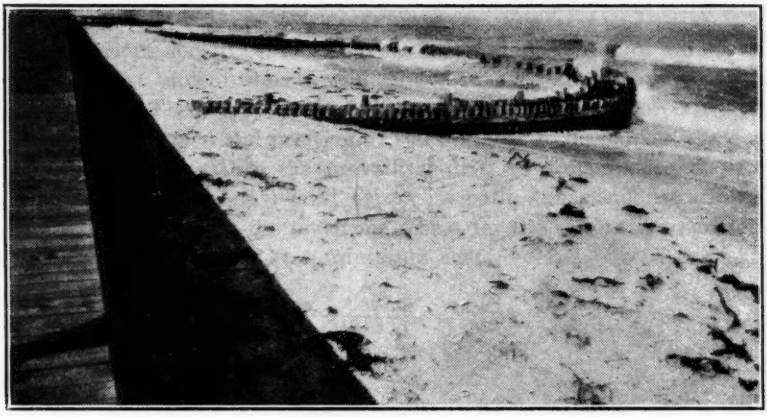


FIG. 28.—BEACH HAVEN, N. J. BULKHEAD TO PRESERVE BEACH. BUILT IN 1916; PHOTOGRAPH TAKEN IN 1920.

FIG. 29.—LONG BEACH, N. J. HOOKED JETTY HAS BURIED ITSELF AND EXTENDED SHORE LINE. MAN STANDING ON BURIED JETTY. PHOTOGRAPHED OCTOBER 31, 1923.

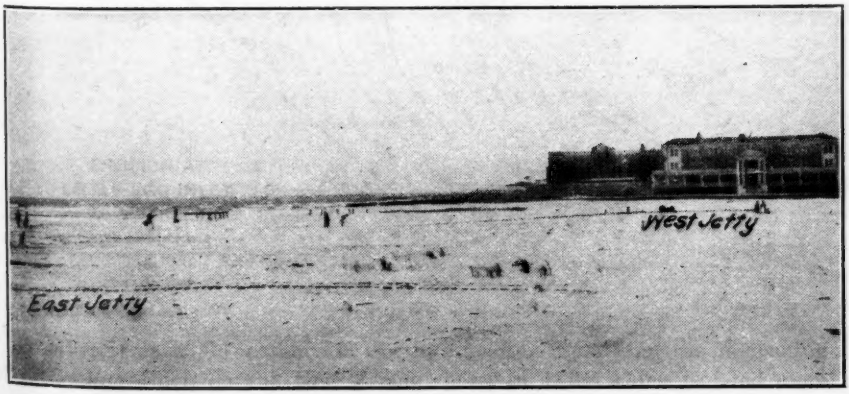


FIG. 30.—ROCKAWAY, N. Y. EFFECT OF TWO JETTIES IN RECLAIMING TWENTY-EIGHT ACRES. JETTIES BUILT IN 1912.

instances at various localities, aggregating forty or more, all of which have withstood the most violent storms which have ravaged these coasts, with no cost for repairs. These unprecedented results have been secured by such modifications in the trace and position of the works as to decompose the direct advance on shore and utilize the prevailing drift by impounding it within or behind permeable barriers placed across its path, thus demonstrating the practicability of controlling the maritime forces for the restoration and maintenance of riparian interests, at a minimum cost.

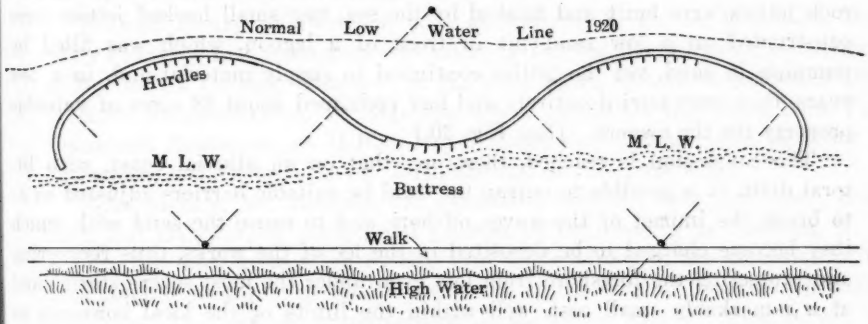


FIG. 31.

This record of the efficiency of the offshore barriers would be incomplete without citing the results of the buttress built at Beach Haven, N. J., in 1916, which covered about 900 ft. of beach and has protected more than 2 000 ft., with no cost for repairs, as it has not been damaged by violent storms nor by the ice, which has covered it. (See Figs. 28 and 31.)

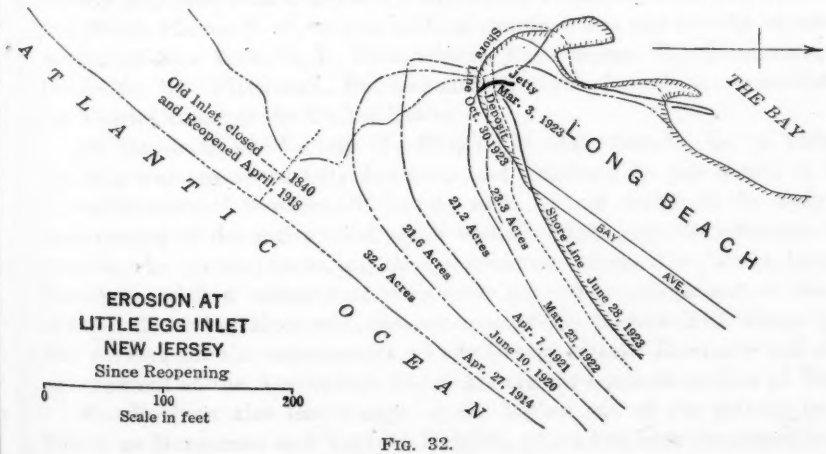


FIG. 32.

Another remarkable instance is worthy of note. Following the re-opening of Little Egg Inlet, on the New Jersey shore, in 1918, which isolated more

than 3 miles of coast under development, and caused considerable loss at the end of Long Beach, it was necessary to construct a hooked jetty at the point, during the winter. The jetty was 304 ft. long, and terminated in water 15 ft. deep; it was completed March 3, 1923, at a total cost of $10 000, with the result shown in Figs. 29 and 32, indicating that it has buried itself and extended the high-water line more than 50 ft. beyond its outer end, after one of the most severe storms in years.

Again, on the south coast of Long Island, where heavy revetments and rock jetties were built and flanked by the sea, two small hooked jetties were constructed on a low sand-spit in front of a lagoon, which was filled by pumping in sand, yet the jetties continued to supply material and, in a few years, they were buried entirely and had reclaimed about 28 acres of valuable property for the owners. (See Fig. 30.)

The conclusion is reached, therefore, that on an alluvial coast, with littoral drift, it is possible to entrap the sand by suitable barriers adjusted so as to break the impact of the waves offshore and to cause the sand with which they become charged to be deposited in the lee of the works, thus recovering and protecting the foreshore from loss even under the most severe storms and at a remarkably small cost, well within the limits of the local boroughs or riparian owners.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM HENRY BALDWIN, M. Am. Soc. C. E.*

DIED MARCH 6, 1917.

William Henry Baldwin was born at Nashua, N. H., on March 10, 1842. After receiving his preparatory education, he entered the Scientific Department of Dartmouth College, Hanover, N. H., from which he was graduated in 1862.

After his graduation, Mr. Baldwin served in the 1st Regiment, New York Volunteer Engineers, at Hilton Head, Charleston Harbor, and in front of Petersburg and Richmond, Va., until the end of the Civil War.

Subsequently, Mr. Baldwin was associated with the firm of Cornell, Bradford, and Baldwin, in the general practice of engineering at Yonkers, N. Y. Among the work undertaken by this firm was a survey of the extension of the 10th Avenue base line to the northern line of the City of Yonkers, including transverse lines and the monumenting of the entire survey. This survey has been of valuable assistance in the municipal development of the city. A City Atlas of Yonkers was also compiled by the firm from numerous surveys and other data, and has been used as a basis for succeeding similar publications.

Mr. Baldwin was also engaged on the location and construction of part of the New York City and Northern Railroad for which he located the line from High Bridge to Lake Mahopac, and had charge of grading and track-laying in the Middle Division from Tarrytown, N. Y., to Croton Lake, about 10 miles.

In 1880, Mr. Baldwin became associated with the late Col. George E. Waring in the design and construction of sewerage and water-works systems in various Southern States, including, Memphis, Tenn., Norfolk, Va., and Buffalo and White Plains, N. Y., and in making examinations and reports on sewerage systems of New York, N. Y., Philadelphia, Pa., Chicago, Ill., Cincinnati, Ohio, Louisville, Ky., Pittsburgh, Pa., and smaller places for use in connection with the Tenth Census of the United States.

On the death of William W. Wilson, M. Am. Soc. C. E., in 1887, Mr. Baldwin was appointed City Engineer and Engineer for the Board of Water Commissioners of Yonkers, N. Y. As such, he had charge of the design and construction of the entire trunk sewer system of that city; the extension of the water-works system, including the construction of the Fort Field Receiving Reservoir, with a capacity of 60 000 000 gal.; the enlargement of the Lake Avenue Receiving Reservoir; the erection of the Nodine Hill Water Tower; and surveys for the enlargement of the Grassy Sprain Reservoir and for the development of the Nepperham River through the business section of Yonkers.

Mr. Baldwin also had charge of the laying out of the private property known as Morsemere and Yonkers Heights, which has been developed into one of the finest residential sections of Yonkers. He also laid out a vast area of

* Memoir prepared by Frank R. Chadeayne, Esq., Yonkers, N. Y.

rough country for the Sullivan Country Club and constructed several miles of roads to open that section.

In 1908, Mr. Baldwin was appointed Deputy City Engineer of Yonkers under the late Samuel L. Cooper, M. Am. Soc. C. E., and after Mr. Cooper's death in 1913, was retained as Acting City Engineer until 1914, when he was again appointed Deputy City Engineer which position he held until his death on March 6, 1917.

He served as a Deacon in the Warburton Avenue Baptist Church of Yonkers, and was also a member of the Masonic Fraternity.

Mr. Baldwin was elected a Member of the American Society of Civil Engineers on June 6, 1888.

CHARLES NEWTON LITTLE, M. Am. Soc. C. E.*

DIED AUGUST 31, 1923.

Charles Newton Little, the son of Charles and Susan Little, was born on May 19, 1858, at Madura, Southern India, where his parents were stationed as missionaries, having dedicated their lives to this work, as earnest Puritan Christians.

Charles Newton Little received his early education in Connecticut and Nebraska. Coming to Lincoln, Nebr., in the Sixties, he subsequently entered the University of Nebraska from which he received the degrees of A. B. in 1879 and A. M. in 1884. After graduate work at Yale University, he received the degree of Ph. D. in 1885.

Mr. Little was Instructor in Mathematics and Civil Engineering at his Alma Mater from 1880 to 1884; Associate Professor of Civil Engineering from 1885 to 1890; and Professor of Engineering from 1890 to 1893. From 1893 to 1901, Dr. Little was Professor of Mathematics at Leland Stanford, Jr., University, in California. In 1899 and 1900, on leave of absence, he studied at the Universities of Göttingen and Berlin, in Germany.

In 1901, under the ægis of an exceptional education, Dr. Little resigned from the Faculty of Leland Stanford, Jr., University, and went to the University of Idaho as Professor of Civil Engineering. Here, in 1911, he became Dean of the College of Civil Engineering, to which position he gave his inspirational endeavor until the close of the 1922-23 year, when he announced his retirement, at the age of 65 years.

Shortly afterward, partly on account of failing health, but still responsive to the lure of his profession, he removed to Berkeley, Calif., to pursue further mathematical investigations. He died suddenly on the afternoon of Friday, August 31, 1923, while engaged in the subtleties of some integrating equations.

Dr. Little's thesis for his doctor's degree was based on research work in the Theory of Knots. This early accomplishment attracted the attention, among others, of Dr. Tait, of the University of Edinburgh, also a worker in the same

* Memoir prepared by J. C. Ralston, M. Am. Soc. C. E.

field. It was at the suggestion of Dr. Tait that Dr. Little continued, as an avocation, his work on this subject. His papers were published in the *Transactions* of the Royal Society of Edinburgh and in those of the Connecticut Academy as early as 1885.

Dr. Little's work on this subject, as also that of other mathematicians, is perhaps only vaguely understood and poorly appreciated. Derrick M. Lehmer, Professor of Mathematics, and Editor of the *University of California Chronicle,* speaking of the Theory of Knots and of Dr. Little's research therein, expresses a doubt as to "whether a popular account could be made sufficiently intelligible to any save those who are specialists in Analysis Situs". He adds further:

"Of the importance of those investigations there need be no question, but just how they may some day connect up with other important mathematical fields no one can foretell. Problems connected with Knots, that is, with curves in space which pass through themselves after the manner easily shown by the knotted string, but difficult to analyze analytically, are doubtless closely connected with the Projective Theory of Space Curves and with Two-Spreads in Four-Space. It is an attractive field, but sooner or later, more work will be done in it for which Professor Little's researches will be invaluable."

In 1923, when Dr. Little went to Berkeley he was associated with two scientists also interested in the Theory of Knots, and, consequently, he found renewed enthusiasm in this research work; for there are few men in the world who know much about the fundamental propositions of knots, their types, orders, and classifications, or of the space curves and special spherical developments which the knotted string may be made to generate.

His ideals of teaching were singularly wholesome and free from the Pharisaical taint which a long career of Pedagogy sometimes engenders. Happily, he was free from the spirit of intolerance; he esteemed his young men not as inferiors, but as fellow students engrossed in the great adventure of seeking truth. He invited them to study, using terms of gentleness; he convinced them by obvious processes rather than by empiric dicta. His habits of straight thinking guided his feet away from the ephemeral "isms" of the present day, and his demand for thoroughness in life and study, coupled with a tolerant and gentle Christian spirit, are some of those purer jewels which the world delights to hold sacred.

Dr. Little was a Fellow of the American Association for the Advancement of Science, a member of the Society for the Promotion of Engineering Education, of the American Association of University Professors, and also of the Honorary Society of Sigma Xi. He was a member of the Congregational Church, devout, reverent, manly. His earnestness of purpose, together with his broad sympathy and interest in others, endeared him to a large circle of friends, especially to the members of the various University faculties with which he had been connected, and to the student bodies.

While Dr. Little was Associate Professor of Civil Engineering at the University of Nebraska, he was married to Miss Emma R. Funke, at Lincoln, on August 5, 1886, who, after thirty-seven years of loyal and inspiring companion-

ship, survives to mourn the loss of a devoted husband, an accomplished gentleman, and an earnest, sincere, high-minded Christian.

Dr. Little was elected a Member of the American Society of Civil Engineers on July 6, 1920.

ADELBERT FRANKLIN PARKER, M. Am. Soc. C. E.*

DIED OCTOBER 13, 1923.

Adelbert Franklin Parker was born at Carver, Mass., on December 14, 1858. He attended the Elliott High School, in Boston, Mass., until 1873, when in search of adventure he went to Utah. In Salt Lake City and Tintic, he studied assaying from 1873 to 1875. At Ophir and Jacob City, during 1875 to 1876, he conducted an Assay Office, devoting his evenings to the study of surveying. From 1877 to 1888, he made rapid advances in his knowledge of surface and underground surveying and the building of irrigation canals.

In 1889, Mr. Parker was made Draftsman and Superintendent of Construction for G. A. d'Hemecourt, of Ogden, Utah. In 1890, he was placed in full charge of the construction of the power house of the Ogden Street Car Company, and also superintended the construction and installation of the steam and electrical machinery.

From 1891 to 1893, he was head of the firm of Parker, Doxey, and Eastman, Architects, with offices at Ogden. During the latter part of 1893, he served as Topographer and Draftsman on the survey for the Ogden Reservoir and Irrigation Works, and, in 1894, he surveyed all the roads and made ownership plats of Summit County, Utah.

During these years, Mr. Parker's reputation was steadily increasing for, unlike many pioneers, this Massachusetts man was not content with the fruits of practice and experience only. As he worked, he learned from day to day the value of other men's experience as recorded in books. It is due in part to his wide reading that Mr. Parker was invited to design and build in many neighboring States, for example, in Oregon, where he worked for the Grant Gold Mining Company; in Yerrington, Nev., where he served in 1897 as Engineer for the Boston-Nevada Gold and Copper Mining Company; in Butte, Mont., where, in 1898, he was in the City Engineering Department; in Blackfoot, Idaho, where he was Constructing Engineer, from 1899 to 1900, for the American Falls Canal and Power Company; and in Elko, Nev., and Evanston, Wyo., for which cities, he designed sewerage systems.

In the autumn of 1898, Mr. Parker became City Engineer of Ogden, which position he held until 1909. Although nearly $2 000 000 was expended under his supervision for Ogden and the surrounding country, this vast and varied work did not consume all his energy, for during the same period he located and designed the West Cache Canal System in Cache County, Utah, and the Oneida Irrigation District Canal, Cache Valley, in addition to much other

* Memoir prepared by G. A. Dickson, M. D., Ogden, Utah.

outside work. From 1909 until his death, Mr. Parker was engaged in private practice as a Consulting Engineer with offices at Ogden.

On June 15, 1886, he was married, in Ogden, to Miss Pamela Mason who, with six children, four boys and two girls, survives him.

Mr. Parker was a prominent member of many Masonic orders, including the Blue Lodge and the Shriners. In this fraternal work, as in all other social relations, he made hosts of friends, because of his shrewd, kindly disposition and his exceptionally well stored mind. After his death on October 13, 1923, and in accordance with his wishes, his library was given to the Carnegie Library of Ogden.

The semi-arid region to which he came in 1873 is now green, fertile, and flower-embroidered. The many hundred flumes, pipes, ditches, and irrigation canals of his building continue to bear witness to the beauty of his mind and spirit and, "where the living waters flow, where the roses bud and blow", his life and work will not be forgotten.

Mr. Parker was elected a Member of the American Society of Civil Engineers on August 31, 1909.

VICTOR HERMAN REINEKING, M. Am. Soc. C. E.*

DIED OCTOBER 14, 1923.

Victor Herman Reineking was born in Franklin, Wis., on August 7, 1884. He attended the public schools and the High School of Sheboygan, Wis., and was graduated from the University of Wisconsin with the degree of Bachelor of Science in Civil Engineering, in the Class of 1908, having entered the University in the Class of 1906.

From August, 1905, to April, 1906, Mr. Reineking was engaged as Chief of Field Party for the U. S. Geological Survey, on water-power surveys in Northern Wisconsin. From April, 1906, to October, 1907, and from May, 1908, to September, 1913, he was employed as Assistant Engineer with Daniel W. Mead, M. Am. Soc. C. E., in charge of field work on water-power investigations, and on the design of dams, power-houses, and other hydraulic structures. He was Resident Engineer on the construction of the dam of the Southern Wisconsin Power Company, at Kilbourn, Wis., and of the clamps at High Falls and Johnson Falls, Peshtigo River, Wisconsin.

In September, 1913, Mr. Reineking went to Portland, Ore., where he engaged in private engineering practice in partnership with Leroy F. Harza, M. Am. Soc. C. E., with whom he collaborated in 1914 in a report on the water-power possibilities of the Columbia River at Celilo Falls for the United States Reclamation Service and the State of Oregon.

From this time until his death in 1923, Mr. Reineking was engaged on many water-power and irrigation projects, both in a consulting capacity and as engineer in responsible charge. In 1914 and 1915, he was in charge of the reconstruction of the Elwha River Dam and Power Plant for the Olympic

* Memoir prepared by Fred F. Henshaw, M. Am. Soc. C. E.

Power Company, near Port Angeles, Wash., a difficult repair job of a dam which had failed by the washing out of its foundation. In 1915 and 1916, he was Division Engineer in charge of the construction of canals and structures for the West Okanogan Valley Irrigation District, and, in 1917 and 1918, Resident Engineer in charge of the Ochoco Hydraulic Fill Dam, Ochoco Irrigation District, near Prineville, Ore. He was Assistant Engineer to Baar and Cunningham from May, 1919, to 1921, in charge of investigations for several irrigation projects.

From 1921 until his death, he maintained an office as Consulting Engineer at Portland, Ore., specializing in irrigation and hydro-electric work. Among the projects on which he prepared reports, but which had not reached the construction stage, were the proposed dam on Columbia River at Umatilla Rapids and the irrigation system for the Dufur Orchards, the largest single orchard in the United States, as well as projects for power development on the Deschutes River.

Mr. Reineking was married on June 1, 1909, to Grace E. Strickland, of Sheboygan, Wis. Mrs. Reineking and three children, Richard, Elizabeth, and Mary Catherine, survive him.

Early in July, 1923, Mr. Reineking went to the Mayo Hospital, at Rochester, Minn., where after a period of treatment he underwent a serious operation in September. He returned to Milwaukee, Wis., to be with his father, Dr. Herman Reineking, and died at the Milwaukee Hospital.

Mr. Reineking was a member of the City Club of Portland and, at the time of his death he was Secretary of the Oregon Chapter of the American Association of Engineers.

He was an engineer of marked ability and thoroughness and, although unassuming, was highly regarded both by members of the Engineering Profession and by his many clients.

Mr. Reineking was elected a Member of the American Society of Civil Engineers on September 12, 1921.

LOUIS EDWARD STROTHMAN, M. Am. Soc. C. E.*

DIED MAY 7, 1922.

Louis Edward Strothman was born in Milwaukee, Wis., on January 10, 1879, and spent his entire life in that city. He attended the public schools, including the South Side High School, from which he was graduated with the Class of 1896. He also attended St. John's Military Academy, at Delafield, Wis.

In 1893, he made his initial step in the business world in the employ of the Filer and Stowell Company, with which he was engaged as a Draftsman until the latter part of 1894. From 1896 to 1899, Mr. Strothman was connected with the Vilter Manufacturing Company and, from 1899 to 1902, with the Nordberg Manufacturing Company. During all this time spent in the Drafting-Rooms

* Memoir prepared by Messrs. A. C. Flory and G. L. Kolberg, Milwaukee, Wis.

of these firms, his purpose was to acquire experience. With this same object in view he entered the employ of the Allis-Chalmers Company in 1902 as a Draftsman in the Pumping Engine Department. A few years later, the Company sent him out as an Erecting Engineer and after he had acquired experience in this work, he was made Engineer in Charge of Centrifugal and Screw Pumping Engine Design.

In 1908, Mr. Strothman was appointed Assistant Manager of the Pumping Engine and Hydraulic Turbine Departments, in which position he was active until the re-organization of these Departments in 1910 when he became Engineer in Charge of the Pumping Machinery Department, which, at that time, handled all types of pumps made by the Company, namely, reciprocating, centrifugal, and screw types. In 1911, Mr. Strothman was appointed Manager and Chief Engineer of the Pumping Engine Department and, in December, 1915, Manager of the Steam Turbine Department, in addition to his other duties. On August 15, 1919, he resigned this office to become Vice-President and General Manager of the Richardson-Phenix Company, which position he held at the time of his death.

In his business life, Mr. Strothman always evinced a stanch determination and tireless energy, and he had established an enviable reputation for earnest, painstaking work and co-operation. As an official of the Richardson-Phenix Company, his work was of inestimable value to the enterprise.

Mr. Strothman had an extensive and varied engineering experience and, in addition to being well known in the National engineering societies, was personally known to many prominent engineers in all the larger cities from coast to coast. In 1909, he was elected a member of the American Society of Mechanical Engineers and took an active interest in the affairs of that organization during the last decade of his life, serving in various offices. In 1915 and 1916, he was Chairman of the Milwaukee Section. In 1916, he was appointed a member of the Board of Directors of the Organization for National Preparedness for the State of Wisconsin, and on the recommendation of the President of the Society was also appointed an Associate Member of the Naval Consulting Board by the Hon. Josephus Daniels, then Secretary of the Navy. In 1918, Mr. Strothman was made a member of the main Committee on Power Test Codes of the American Society of Mechanical Engineers and was elected Chairman of the Sub-Committee on Displacement Pump Tests. In 1919, he was appointed by M. E. Cooley, M. Am. Soc. C. E., then President of the American Society of Mechanical Engineers, to represent that Society, with Mr. Cooley, on the National Industrial Conference Board. In December, 1921, Mr. Strothman was elected Vice-President of the American Society of Mechanical Engineers. He was also a member of the American Water Works Association and the Engineering Society of Milwaukee, of which he was President in 1916 and 1917.

Mr. Strothman was married on May 11, 1899, to Miss Galena Pennell, who, with four children, Edmund Pennell, Dorothy Louise, Thomas Henry and Robert Louis, survives him.

General Otto H. Falk, President of the Allis-Chalmers Manufacturing Company, and a close personal friend of Mr. Strothman, pays him the following tribute:

"During the nearly twenty years of his connection with the Allis-Chalmers organization, Louis E. Strothman advanced from the position of Draftsman to that of Manager of the Steam Turbine and Pumping Engine Departments. He was recognized, not only in our organization, but throughout the Engineering Profession, as an able engineer and capable executive. His spirit of cooperation as well as his kindly personality endeared him to all of those with whom he came in contact."

Mr. Strothman was elected a Member of the American Society of Civil Engineers on December 11, 1911.

ISAAC WINSTON, M. Am. Soc. C. E.*

DIED DECEMBER 7, 1923.

Isaac Winston was born near Tuscumbia, Ala., on September 7, 1853. He was educated in private schools from 1864 to 1870, and the Virginia Military Institute from which he was graduated in 1873. He always retained an affectionate regard for that institution, and his loyalty was recognized at the Fiftieth Anniversary of his Class in June, 1923.

After leaving college, Mr. Winston was for two years a member of the United States Engineer Corps on the construction of the Muscle Shoals Canal.

On July 18, 1878, he was appointed to the U. S. Coast and Geodetic Survey as a Recorder and was made an Aid on July 10, 1879, a Sub-Assistant on July 1, 1884, and an Assistant on February 5, 1891. On July 1, 1917, this designation was changed, and he was commissioned by President Wilson as Hydrographic and Geodetic Engineer with the relative rank of Lieutenant-Commander in the U. S. Navy.

Mr. Winston was placed on the retired list on May 18, 1920, having passed the retirement age of 64 years. On an order effective May 18, 1920, he was recalled to active duty. On November 10, 1921, he was placed on the inactive list on account of failing health, but on April 10, 1922, he was recalled to active duty. On January 16, 1923, he was, however, again placed on the inactive list on account of failing health.

During his long career with the Coast and Geodetic Survey, Mr. Winston was employed in various parts of the United States on astronomical work, reconnoissance, triangulation, magnetic work, and precise leveling. For seven years, he served as Editor of the publications of the Survey. His last active duty was in charge of the Field Station of the Survey at New York, N. Y.

He died at his residence in New York, N. Y., after a long illness, on December 7, 1923. The funeral was held from St. Paul's Roman Catholic Church, Washington, D. C., on December 10, 1923, the honorary pallbearers being officers of the U. S. Coast and Geodetic Survey.

* Memoir prepared by E. Lester Jones, Assoc. M. Am. Soc. C. E.

Mr. Winston was married on September 19, 1882, to Anna Maria Otis who, with a son, survives him.

He was a man of great energy and activity, deeply interested in his profession, and displaying always a high sense of duty and loyalty to the service. He was noted for his strict integrity in all relations of life. Of a naturally kind and sympathetic disposition, he was devoted to his family and friends, and always ready to render assistance in time of sorrow or affliction.

Mr. Winston served as a Delegate from the United States to the International Geodetic Association, 13th Conference, in 1900; and was a member of the American Association for the Advancement of Science, Philosophical Society, Washington Society of Engineers, and Cosmos Club.

The following tribute was paid Mr. Winston by the Director of the U. S. Coast and Geodetic Survey, before the Commissioned Officers of the Bureau on December 10, 1923:

"There is one who has been uppermost in my mind to-day. I feel a special desire to mention the passing on of Mr. Isaac Winston, whose funeral I attended to-day and who for nearly half a century was an officer of the Coast and Geodetic Survey. My great regret is that you younger men have not had the privilege of knowing him. I am sure I reflect fully the feeling of his other associates, when I say that the privilege of having been with him and of having had his counsel has not only been invaluable to me personally, but that it has been a wonderful help in maintaining the high standards for which the Bureau stands. Those who have known him are fortunate; to those who have not had this pleasure, I say with perfect propriety that Mr. Winston reflected the ideal man. He was ever ready to do his part, day or night, in his outstanding loyalty to his co-workers, the Bureau, and the country. His Christian spirit was a vital part of his daily life. It was evidenced in his daily efforts, thereby reflecting to those around him the unselfish kindly man that he always was.

"If we can hold him up as an example and follow in his footsteps, there is no question as to the proper report of our stewardship in the years to come.

"Personally I shall always cherish the privilege I had in knowing him so well. In the years to come, the monument that he left behind will be a source of pride and comfort to those who will carry on."

Mr. Winston was elected a Member of the American Society of Civil Engineers on July 11, 1921.

JAMES MARCUS BANDY, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 23, 1911.

James Marcus Bandy was born near Newton, N. C., on January 8, 1850. He was educated at the common schools of Catawba County and, later, at Rutherford College, Trinity College, Durham, N. C., and the University of Virginia, from which he received the degrees of Bachelor of Arts, Master of Arts, and Civil Engineer.

From 1885 to 1893, he was Professor of Mathematics and, during the last four years of this period, Professor of Civil Engineering, at Trinity College.

* Memoir prepared by Bruce Craven, Esq., Trinity, N. C.

In 1893, Professor Bandy was appointed Chief Engineer on the Page Railroad (now the Norfolk Southern Railroad), from Troy to Mt. Gilead, N. C., and from 1896 to 1898, he served as Chief Engineer of the Carolina, Fayetteville, and Northern Railroad Company.

In 1898, Professor Bandy was elected City Engineer of Greensboro, N. C., and, as such, designed the water-works plant for that city as well as the sewerage system. He was also engaged on special work for various municipalities in North Carolina, including the 400-ft. Deep River Bridge in Randolph County.

From 1906 until his death in 1911, Professor Bandy was engaged in the private practice of engineering, as a Consulting Hydraulic and Sanitary Engineer, with an office at Greensboro.

Professor Bandy was first married to Miss Martha Jane Leonard, of Lincoln County, North Carolina, and after her death to Mrs. Sallie Joyner, of Laurinburg, N. C., who, with three daughters and three sons by his first wife, survived him.

He was a man of brilliant mind and had many friends. A lover of good music, he played the violin with ability. As an engineer, his work will be remembered for the excellence of his construction operations for various municipalities, and, as a devoted student of the higher mathematics, for the many excellent mathematicians whose minds he helped to train during his professorship at Trinity College. He will also be remembered by the older students of Trinity College for his humor and his generosity.

Professor Bandy was elected an Associate Member of the American Society of Civil Engineers on February 4, 1903.

FRITZ CARL ANDERS GEORG BERGENGREN, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 8, 1904.

Fritz Carl Anders Georg Bergengren, the son of George and Elisabeth Netsler Bergengren, was born in Helsingborg, Sweden, on May 24, 1863. His father was a physician in the service of the Swedish Government.

He entered the Military Academy of Carlberg, Sweden, in 1883, and on the completion of his course, in 1885, was commissioned a Lieutenant in the Royal Vendes Artillery Regiment.

In 1886, Mr. Bergengren became Assistant Chemical Engineer with the Skane Sulphuric Acid and Superphosphate Manufacturing Company, but gave up this work in 1888 and came to the United States. He secured employment as a Draftsman in the Patent Office of Brown and Griswold, in New York, N. Y.; later, in 1889, he accepted a position with the Pencoyd Iron Works, at Pencoyd, Pa., first, as a Draftsman and, afterward, as Assistant Engineer in the Bridge Department. While with this Company, he had charge of the force that prepared the drawings for the bridge constructed by the Pennsylvania Railroad

* Memoir prepared by Thomas Earle, M. Am. Soc. C. E.

Company across the Delaware River, above Philadelphia, Pa., for its Atlantic City Division. In 1894, Mr. Bergengren became a citizen of the United States.

In the spring of 1896, he entered the service of the Pennsylvania Steel Company, at Steelton, Pa., as Chief Draftsman of the Bridge and Construction Department, which position he held until 1901. During that time, plans for a number of important bridges were prepared in the Drafting Room of the Company, among others being the Scherzer Rolling Lift Bridge built by the New York, New Haven and Hartford Railroad Company over Fort Point Channel at Boston, Mass.; the Niagara Arch Bridge constructed by the Grand Trunk Railway Company at Niagara Falls, N. Y.; the bridge across the Duluth River from Duluth, Minn., to Superior, Wis.; and the Gokteik Viaduct built for the Burma Railways in Upper Burma, India.

In 1901, with Mr. John C. Stulen, Mr. Bergengren organized the John C. Stulen Company, Engineers and Inspectors, of Harrisburg, Pa., and carried on a successful business until his death on August 8, 1904.

Mr. Bergengren was capable, energetic, and broad-minded. He had an agreeable personality, and gave great promise of becoming widely known in his profession.

He was married on November 26, 1896, to Marion L. Holcomb, of Whitehall, N. Y., by whom he was survived.

Mr. Bergengren was elected an Associate Member of the American Society of Civil Engineers on December 2, 1896.

GEORGE WHITNEY KINNE, Assoc. M. Am. Soc. C. E.*

DIED MARCH 14, 1922.

George Whitney Kinne was born at Ilion, N. Y., on November 6, 1875. His early education was acquired in the Ilion High School and the Fairfield Military Academy. In 1895, he entered the Rensselaer Polytechnic Institute, at Troy, N. Y. In May, 1898, during the Spanish-American War, he was enrolled in Company D, 2d New York Infantry, and was mustered out of service with Company G, 1st Engineers, as a Corporal, in January, 1899. He then resumed his studies at Rensselaer Polytechnic Institute, from which he was graduated with the degree of Civil Engineer in 1900.

Immediately after his graduation, Mr. Kinne entered the service of the West Side Structural Company, of Troy, N. Y., and served as Shop Superintendent for 18 months. In November, 1901, he entered the employ of the American Bridge Company as a Draftsman at the plant in Elmira, N. Y. In 1902, he was transferred to the Erecting Department of the same Company as Assistant Superintendent of Erection, with headquarters at Elmira. In October, 1905, he was transferred to the Philadelphia Office of the Erecting Department, where he was promoted to the position of Assistant Engineer, with supervision of the field operations performed by sub-contractors on the falsework for the Pennsylvania Railroad Bridge, at Havre de Grace, Md.; the

* Memoir prepared by Joseph B. Gemberling, M. Am. Soc. C. E.

Union Station and U. S. Office Buildings, at Washington, D. C.; a bridge on the Florida East Coast Railway, at Stuart, Fla.; and a number of commercial and office buildings in Baltimore, Md.

In 1906, Mr. Kinne was assigned as Assistant Engineer on the reconstruction of the cantilever bridge over the Hudson River at Poughkeepsie, N. Y., in charge of the alterations necessary to the existing structure to accommodate a new center truss. On this work, he directed an average force of 60 men for 22 months, which included undergraduates from several engineering schools, who desired work and experience during the summer vacations. In 1908, he resumed his position in the Philadelphia Office of the Erecting Department of the American Bridge Company, as Assistant Engineer, with supervision of the field work performed by sub-contractors.

In 1909, Mr. Kinne was assigned to the office of the Erecting Department of the Company in Jersey City, N. J., as Assistant Engineer in Charge. During the period from 1909 to 1917, many important bridges and structures were erected under his immediate supervision, namely, the Madison Avenue Draw Bridge, New York City; Honneywell Street Viaduct, Long Island Railway Company; fifty-three steel bridges on the New York, Westchester and Boston Railway; bridges over the Passaic and Hackensack Rivers for the Central Railroad of New Jersey; bridge replacements on the Shore Line Division of the New York, New Haven, and Hartford Railroad; buildings for H. Johns-Manville Company, Mansville, N. J.; the American Brass Company, in the Naugatuck Valley, Connecticut; the Chase Metal Company, Waterville, Conn.; and the Crucible Steel Company of America, Harrison, N. J. He supervised, on an average, twelve bridge and building erection forces, decided methods of erection, ordered the equipment, designed the falsework and rigging, and organized the work.

In September, 1917, he left the employ of the American Bridge Company to enter the service of the U. S. Government as Student Officer at the Engineer Reserve Camp, American University, Washington, D. C. In December, 1917, he was commissioned a Captain of Engineers and was assigned to service under the Director of Military Railways. In March, 1918, he was placed in charge of the erection and operation of twenty gantry cranes on the American docks, at Bassens, France, and, in September, 1918, he was made Depot Engineer Officer at Montierchaume, France, in responsible charge of all engineering material and equipment intended for the Third Army.

In October, 1919, Mr. Kinne was mustered out of the Army and reinstated in the service of the American Bridge Company as Assistant Engineer, with headquarters at Philadelphia, Pa., which position he held at the time of his death. In March, 1920, he was appointed a Major in the Engineer Officers' Reserve Corps.

Mr. Kinne was an example of the highest type of American manhood. He was not only a thoroughly competent and enthusiastic engineer, but a most lovable companion, and by his straightforwardness, sincerity, and honesty of purpose, endeared himself to every one with whom he came in contact. His circle of friends and admirers not only extended throughout the United States, but in foreign lands as well.

Mr. Kinne was elected an Associate Member of the American Society of Civil Engineers on December 5, 1906.

PHILIP RAPP, Assoc. M. Am. Soc. C. E.*

Died June 26, 1922.

Philip Rapp was born in Buffalo, N. Y., on November 9, 1890. Quite early in life he evidenced his limitless determination and capacity for study by obtaining his technical education under conditions that would have proved prohibitive to most young men.

He began his professional life in 1910 as a member of a Survey Party for the New York Central Railroad. Leaving the railroad in the same year, Mr. Rapp joined the Engineering Department of the New York State Highway Department. He remained with this Corps until 1918, except for a short time during 1913, when he took advantage of opportunities to broaden his engineering experience by gaining some insight into office methods, first with Walter McCulloh, M. Am. Soc. C. E., at Niagara Falls, N. Y., and, later, with the State Barge Canal Commission at Lockport, N. Y. Again, during 1916, he obtained leave of absence from the State Highway Department to enter the Field Force of The Donner Steel Company during the construction of the foundations for a new plant at Buffalo.

In 1918, Mr. Rapp entered the employ of the Truscon Steel Company, as a Designing Engineer on all types of concrete buildings. Although he had had no previous experience in this kind of work, he developed such satisfactory results that his ability and aptitude were recognized almost immediately, and he was advanced in the organization at an extraordinary rate. In March, 1919, he was assigned to Cleveland, Ohio, as District Engineer for that district, where he rapidly built up an efficient corps; starting with no assistants at all, he soon developed a large, well balanced staff. In this position, he showed a side of his character hitherto unsuspected, namely, a remarkable ability in the handling of men and the co-ordination of their efforts to produce economical results. He also developed a large personal clientele among those with whom he was thrown in contact. He continued in this position until his death on June 26, 1922.

Among his friends, Mr. Rapp was known as a man of attainments far beyond the ordinary. Professionally, he was remarkable in that he combined profound knowledge of theoretical principles with a practical understanding of work-day methods and commercial limitations. As a man, he was known to be absolutely honest, sincere, and loyal. His word was taken on any professional matter as final evidence. He was a member of Washington Lodge No. 240, F. and A. M., of Buffalo.

On May 1, 1918, Mr. Rapp was married to Mabel Lucy Crowe, at Lockport, N. Y. He is survived by his widow and a son, Philip Crowe Rapp.

Mr. Rapp was elected an Associate Member of the American Society of Civil Engineers on March 12, 1918.

* Memoir prepared by R. D. Kirkwood, Esq., Cleveland, Ohio.